Maryland
Department of
the Environment

Overtopping Protection 101

Scott Bass, P.E.

MDE Dam Safety

Hydrology and Hydraulics

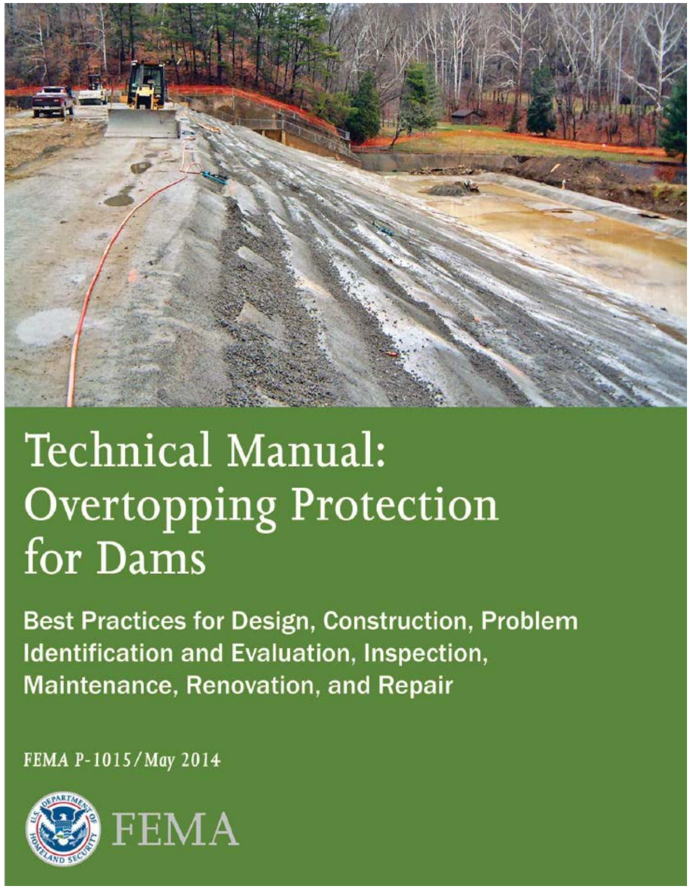
Outline

- Introduction to Dam Overtopping
- Modeling Considerations
- General Considerations
- Design and Construction Considerations
- The Importance of Hiring a Qualified, Experienced Engineer
- Different Overtopping Protection Systems

References – FEMA Overtopping Protection Technical Manual

- https://www.fema.gov/media-library/assets/documents/97888

- Roller Compacted Concrete and Soil Cement
- Conventional or Mass Concrete
- Precast Concrete Blocks
- Gabions
- Vegetative Cover, Turf Reinforcement Mats, Synthetic Turf
- Flow-through Rockfill and Reinforced Rockfill
- Rip Rap
- Geomembrane Liners, Geocells, and Fabric-Formed Concrete



Technical Manual:
Overtopping Protection
for Dams

Best Practices for Design, Construction, Problem Identification and Evaluation, Inspection, Maintenance, Renovation, and Repair

FEMA P-1015/May 2014

FEMA

References - ASDSO Webinars

- https://www.damsafety.org/training-center

Dam Overtopping Protection Systems Part I: On-Demand

Details:
Speaker: Tom Hepler

This is the first of a two-part series on overtopping protection systems for embankment and concrete dams. Many dams today have insufficient spillway capacity and may be overtopped during large floods below the regulatory requirement. In recent years, dam overtopping protection has become increasingly popular as a means to provide additional flood release capacity when more conventional methods such as increasing spillway discharge capacity or reservoir storage are cost prohibitive or impractical. This course will present many of the overtopping protection systems currently in use, describing for each their range of design applications, construction considerations, and potential limitations and risks. Using FEMA's 2014 Technical Manual: Overtopping Protection for Dams as a guide, the two most commonly used systems today, roller-compacted concrete (RCC), and articulating concrete blocks (ACB) will first be presented, followed by various reinforced concrete, turf, and rockfill alternatives for embankment dams.

Dam Overtopping Protection Systems Part 2: On-Demand

Details:
Speaker: Christopher Thornton, Ph.D., Associate Professor, Director, Colorado State University

This is the second of a two-part series on overtopping protection systems for embankment and concrete dams. Many dams today have insufficient spillway capacity and may be overtopped during large floods below the regulatory requirement. In recent years, dam overtopping protection has become increasingly popular as a means to provide additional flood release capacity when more conventional methods such as increasing spillway discharge capacity or reservoir storage are cost prohibitive or impractical. This course will present the process of evaluating protections systems for use in spillway applications. A summary of determining appropriate project hydraulic conditions will be followed by a brief history of prototype research conducted on multiple types of protecting technologies. Results of these tests will be presented in support of design methodologies where new methods for determining project FOS and numerically evaluating hydraulic jump system performance will be recommended. Procedures outlined in NRCS NEH Part 628 Chapter 54, “Articulated Concrete Block Armored Spillways” will

Earthen Dam Overtopping

- Earthen Embankments, typically, consist of highly erodible material when subjected to significant flow depths, for a sustained period of time
  - Earthen embankment are for holding back water, not conveying it.
- Many dams were not originally designed to pass the storm event currently required by their hazard classification.
- Absent overtopping protection, dams can be assumed to sustain significant damage or full breach when subjected to significant overtopping flow.
- Overtopping is the most common failure mode for earthen dam embankments
  - Famous Examples

Vegetal Protection and Hardened Crests Offer Limited Protection

- Well maintained vegetation on uniform slopes, with no irregularities (slope changes, protrusions, defects), can provide <u>some</u> protection during overtopping, but should not be relied upon to prevent full breach. Requires immaculate maintenance.

- Asphalt or concrete crest can provide <u>some</u> level of protection, but should not be relied upon to prevent full breach.

- If overtopping occurs, or is imminent, on an unprotected embankment, best to assume that failure likely to occur, and activate emergency measures.
  - Depth, duration will ultimately drive what happens.

Well Maintained Vegetation

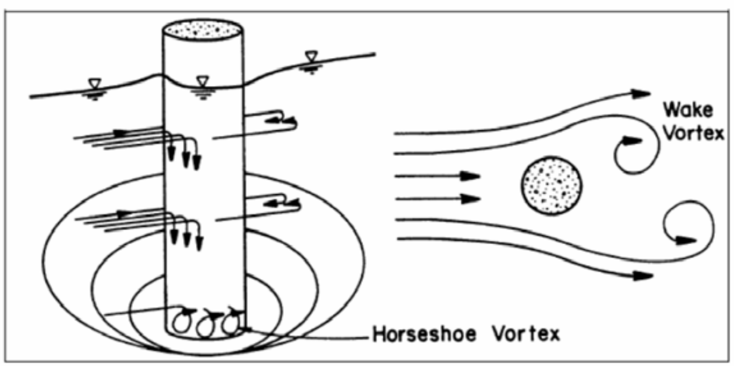

Three Phase Approach

Photos courtesy of Darrell Temple

Breach Process. USDA – ARS Research

1. Failure of Vegetal protection – development of concentrated flow
2. Concentrated flow surface erosion leading to the formation of a vertical, or near-vertical headcut
3. Downward and headward advance of headcut

Local Scour

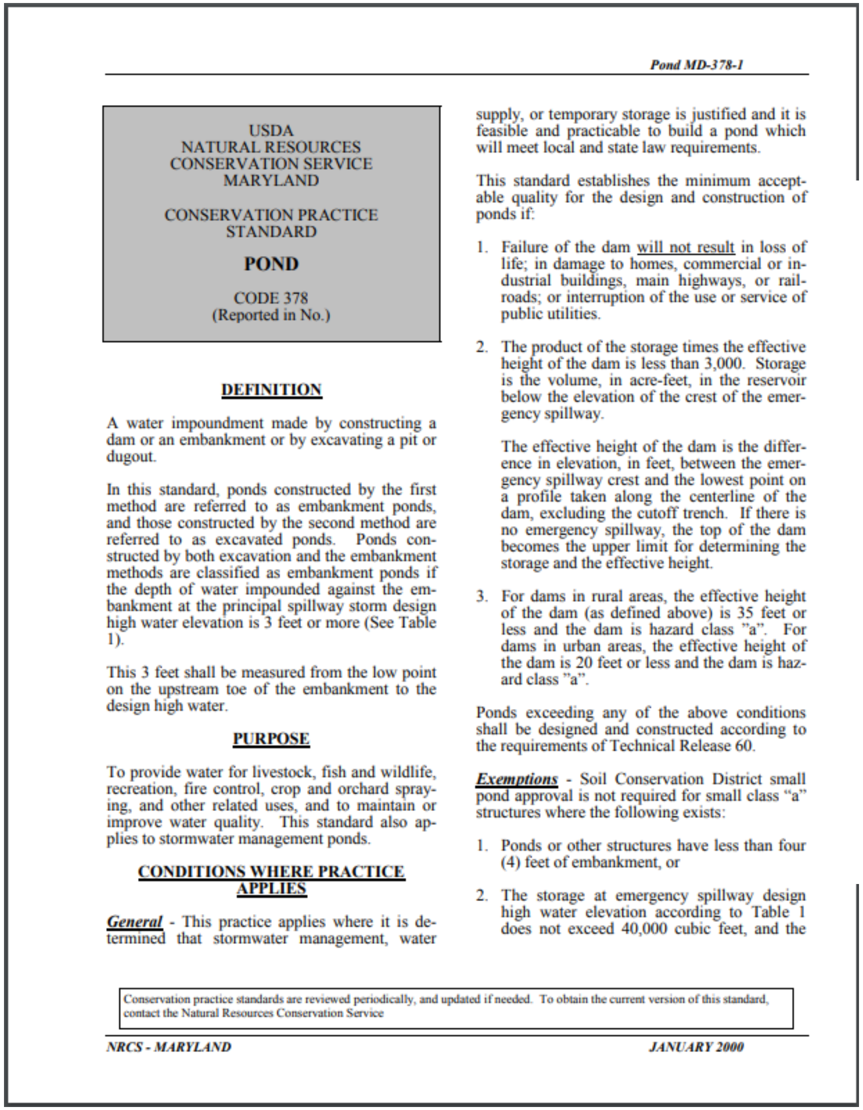


igure 1.2 Schematic Representation of Local Scour at a Cylindrical Pier (Richardson et al., 2001)

Reasons to Consider Overtopping Protection

- MDE Dam Safety has classified dam as “Unsafe – Hydraulically Inadequate”
  - Design Storms
    - Low Hazard – 100-year storm with Freeboard
    - Significant Hazard – ½ PMF with Freeboard
    - High Hazard – PMF with Freeboard
- Hazard creep
- Design storm change
- Hydrologic changes in contributing watershed
- Desire to protect assets

Background



Pond MD-378-1

USDA
NATURAL RESOURCES
CONSERVATION SERVICE
MARYLAND

CONSERVATION PRACTICE
STANDARD

POND

CODE 378
(Reported in No.)

DEFINITION

A water impoundment made by constructing a dam or an embankment or by excavating a pit or dugout.

In this standard, ponds constructed by the first method are referred to as embankment ponds, and those constructed by the second method are referred to as excavated ponds. Ponds constructed by both excavation and the embankment methods are classified as embankment ponds if the depth of water impounded against the embankment at the principal spillway storm design high water elevation is 3 feet or more (See Table 1).

This 3 feet shall be measured from the low point on the upstream toe of the embankment to the design high water.

PURPOSE

To provide water for livestock, fish and wildlife, recreation, fire control, crop and orchard spraying, and other related uses, and to maintain or improve water quality. This standard also applies to stormwater management ponds.

CONDITIONS WHERE PRACTICE APPLIES

General - This practice applies where it is determined that stormwater management, water supply, or temporary storage is justified and it is feasible and practicable to build a pond which will meet local and state law requirements.

This standard establishes the minimum acceptable quality for the design and construction of ponds if:

1. Failure of the dam will not result in loss of life; in damage to homes, commercial or industrial buildings, main highways, or railroads; or interruption of the use or service of public utilities.
2. The product of the storage times the effective height of the dam is less than 3,000. Storage is the volume, in acre-feet, in the reservoir below the elevation of the crest of the emergency spillway.

   The effective height of the dam is the difference in elevation, in feet, between the emergency spillway crest and the lowest point on a profile taken along the centerline of the dam, excluding the cutoff trench. If there is no emergency spillway, the top of the dam becomes the upper limit for determining the storage and the effective height.
3. For dams in rural areas, the effective height of the dam (as defined above) is 35 feet or less and the dam is hazard class "a". For dams in urban areas, the effective height of the dam is 20 feet or less and the dam is hazard class "a".

Ponds exceeding any of the above conditions shall be designed and constructed according to the requirements of Technical Release 60.

Exemptions - Soil Conservation District small pond approval is not required for small class "a" structures where the following exists:

1. Ponds or other structures have less than four (4) feet of embankment, or
2. The storage at emergency spillway design high water elevation according to Table 1 does not exceed 40,000 cubic feet, and the

Conservation practice standards are reviewed periodically, and updated if needed. To obtain the current version of this standard, contact the Natural Resources Conservation Service

NRCS - MARYLAND *JANUARY 2000*

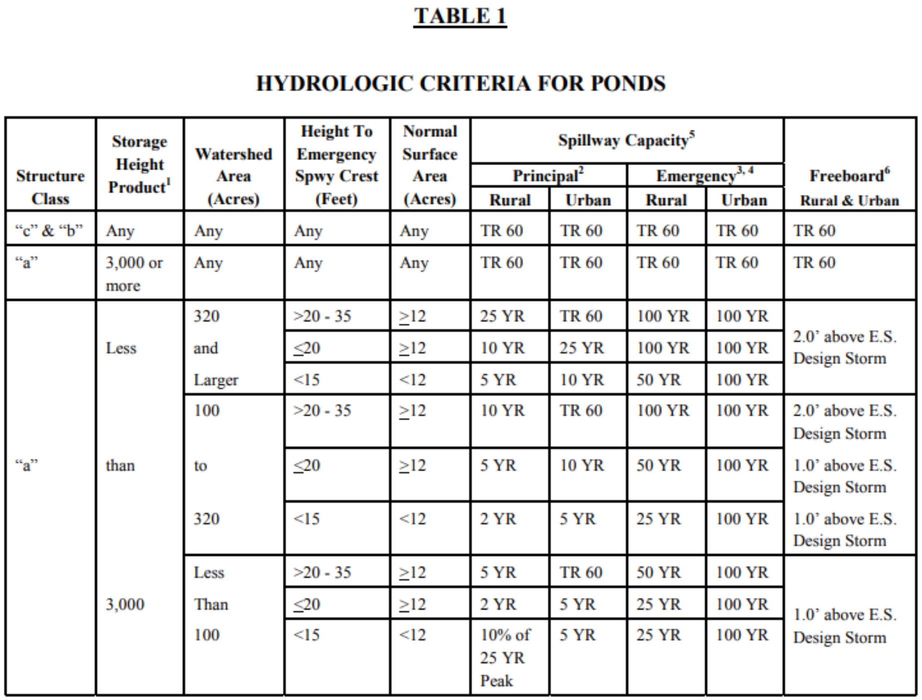
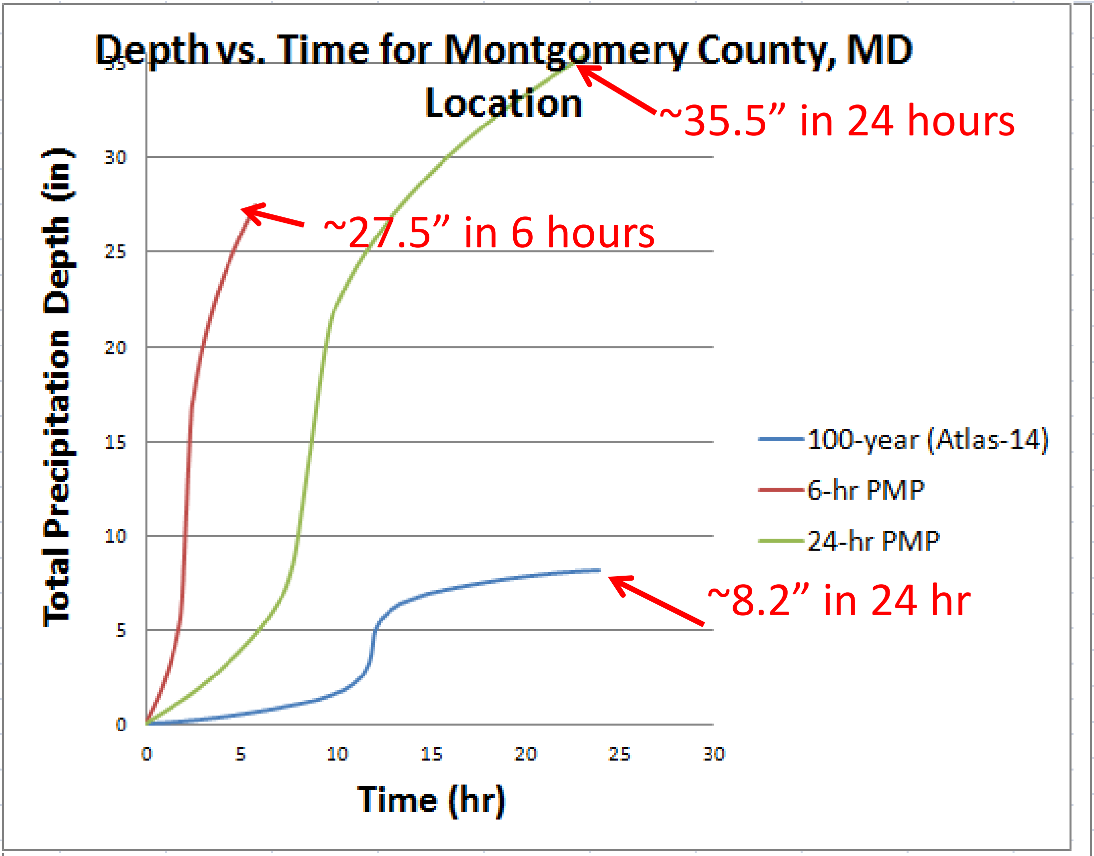


TABLE 1

HYDROLOGIC CRITERIA FOR PONDS

Structure Class	Storage Height Product[1]	Watershed Area (Acres)	Height To Emergency Spwy Crest (Feet)	Normal Surface Area (Acres)	Spillway Capacity[5]				Freeboard[6]
					Principal[2]		Emergency[3, 4]		
					Rural	Urban	Rural	Urban	Rural & Urban
"c" & "b"	Any	Any	Any	Any	TR 60	TR 60	TR 60	TR 60	TR 60
"a"	3,000 or more	Any	Any	Any	TR 60	TR 60	TR 60	TR 60	TR 60
"a"	Less than 3,000	320 and Larger	>20 - 35	≥12	25 YR	TR 60	100 YR	100 YR	2.0' above E.S. Design Storm
			≤20	≥12	10 YR	25 YR	100 YR	100 YR	
			<15	<12	5 YR	10 YR	50 YR	100 YR	
		100 to 320	>20 - 35	≥12	10 YR	TR 60	100 YR	100 YR	2.0' above E.S. Design Storm
			≤20	≥12	5 YR	10 YR	50 YR	100 YR	1.0' above E.S. Design Storm
			<15	<12	2 YR	5 YR	25 YR	100 YR	1.0' above E.S. Design Storm
		Less Than 100	>20 - 35	≥12	5 YR	TR 60	50 YR	100 YR	1.0' above E.S. Design Storm
			≤20	≥12	2 YR	5 YR	25 YR	100 YR	
			<15	<12	10% of 25 YR Peak	5 YR	25 YR	100 YR	

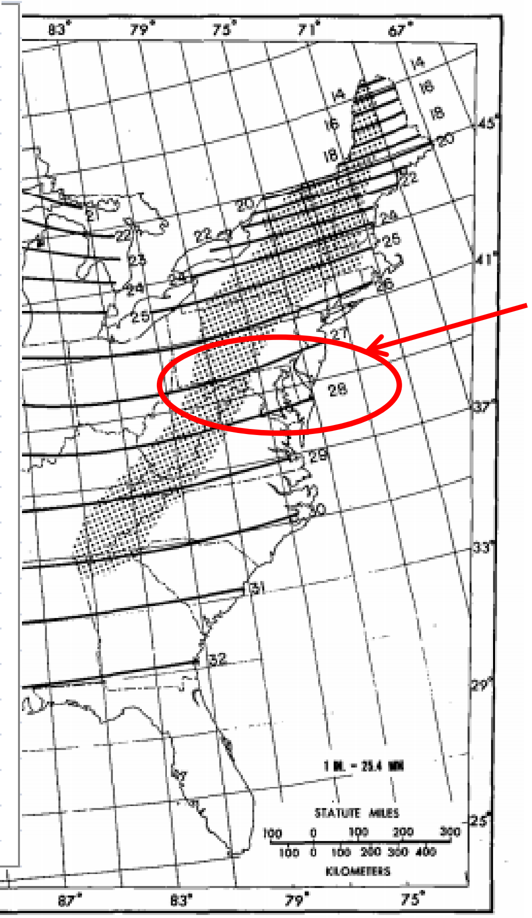
The PMP, continued



~35.5” in 24 hours

~27.5” in 6 hours

~8.2” in 24 hr



Maryland
~27-28.5”
in 6 hours
DA < 10 mi^2

Figure 18.--All-season PMP (in.) for 6 hr 10 mi^2 (26 km^2).

Traditional Methods for Addressing Hydraulic Inadequacy

- Raising Dam
- Lowering permanent pool
- Increase Spillway Capacity
- Addition of auxiliary spillways
- Increase Storage Volume
- Some Combination of these

Considerations that limit traditional methods for addressing inadequacy

- Potential for Downstream Flooding
- Potential for Upstream Flooding
- Property and Topographical Constraints
- Practicality and Cost of other alternatives
- Stakeholder Consideration

Acceptance

- Generally inappropriate for frequent use
  - Should be for storms <100 year storm
- Not to be used as primary spillway
- Other options should be considered
- Design appropriate for Hazard Class, size, flow conditions, embankment makeup, etc.

Type of Overtopping Protection

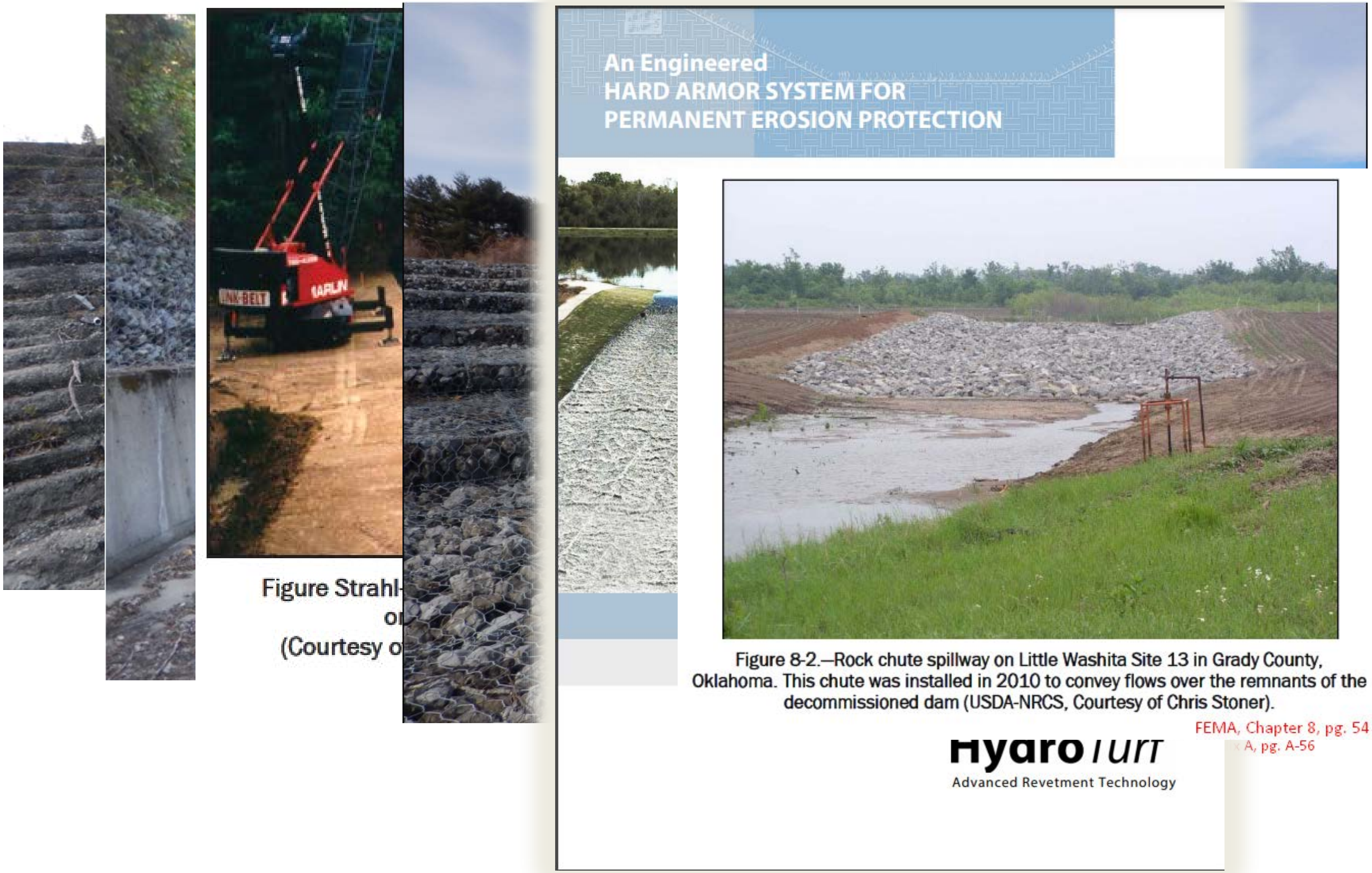


Figure Strahl-
o
(Courtesy o

An Engineered
HARD ARMOR SYSTEM FOR
PERMANENT EROSION PROTECTION

Figure 8-2.—Rock chute spillway on Little Washita Site 13 in Grady County, Oklahoma. This chute was installed in 2010 to convey flows over the remnants of the decommissioned dam (USDA-NRCS, Courtesy of Chris Stoner).

FEMA, Chapter 8, pg. 54
A, pg. A-56

Hydro*Turf*
Advanced Revetment Technology

Failure Modes

Potential Failure Modes for Overtopping Protection

- High velocity flow erodes or removes system
- Uplift pressures lift and remove system
- Erosion initiates at edges of system
- Flow passes beneath system
- Turbulent flow in hydraulic jump damages system
- Differential foundation settlement damages system
- System design capacity is exceeded
- System fails due to poor installation methods
- System fails due to deterioration over time
- Adverse effects on existing dam stability
- FAILURE MAY RESULT IN BREACH OF DAM

Source: ASDSO Dam Overtopping Protection Systems Part 1

Selecting an Overtopping Protection

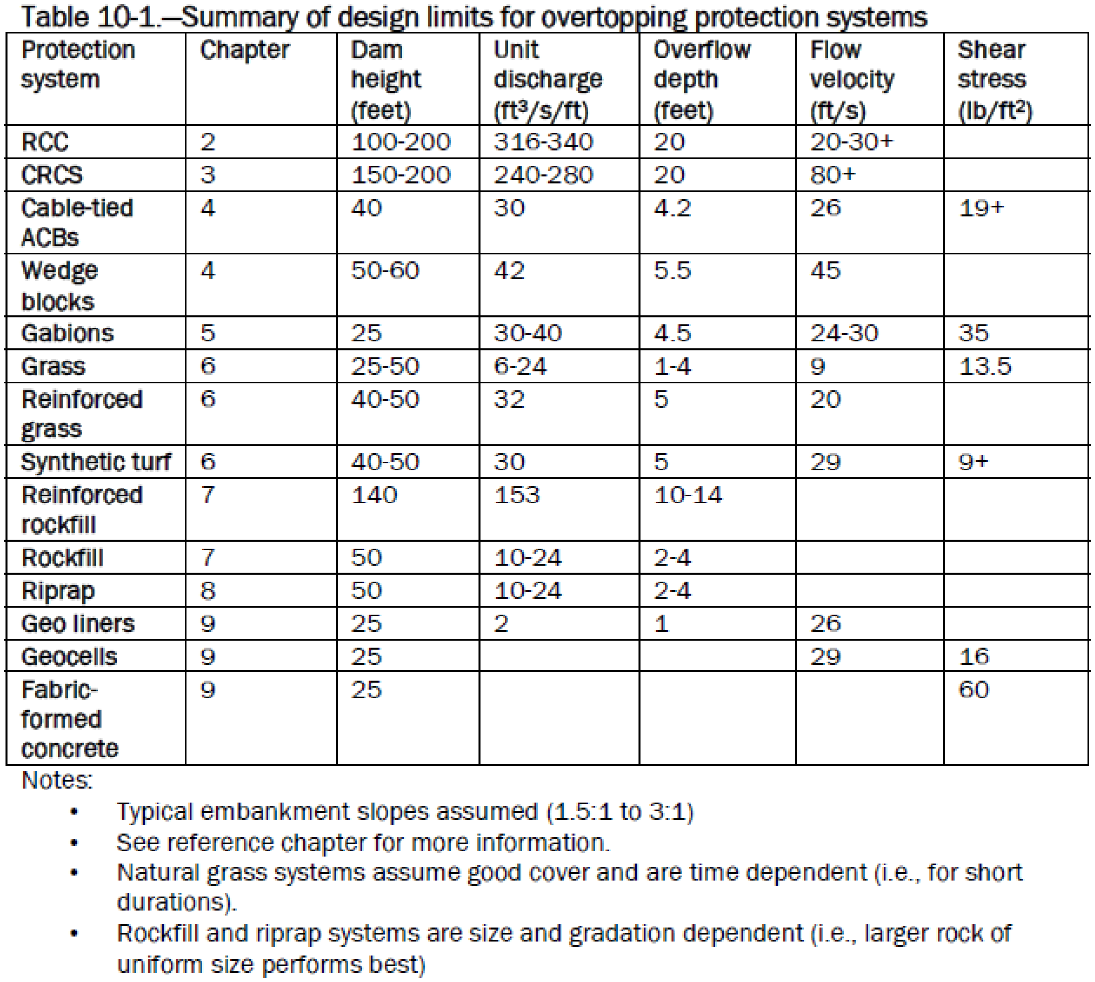


Table 10-1.—Summary of design limits for overtopping protection systems

Protection system	Chapter	Dam height (feet)	Unit discharge ($ft^3/s/ft$)	Overflow depth (feet)	Flow velocity (ft/s)	Shear stress (lb/ft^2)
RCC	2	100-200	316-340	20	20-30+	
CRCS	3	150-200	240-280	20	80+	
Cable-tied ACBs	4	40	30	4.2	26	19+
Wedge blocks	4	50-60	42	5.5	45	
Gabions	5	25	30-40	4.5	24-30	35
Grass	6	25-50	6-24	1-4	9	13.5
Reinforced grass	6	40-50	32	5	20	
Synthetic turf	6	40-50	30	5	29	9+
Reinforced rockfill	7	140	153	10-14		
Rockfill	7	50	10-24	2-4		
Riprap	8	50	10-24	2-4		
Geo liners	9	25	2	1	26	
Geocells	9	25			29	16
Fabric-formed concrete	9	25				60

Notes:

- Typical embankment slopes assumed (1.5:1 to 3:1)
- See reference chapter for more information.
- Natural grass systems assume good cover and are time dependent (i.e., for short durations).
- Rockfill and riprap systems are size and gradation dependent (i.e., larger rock of uniform size performs best)

Flow Characteristics

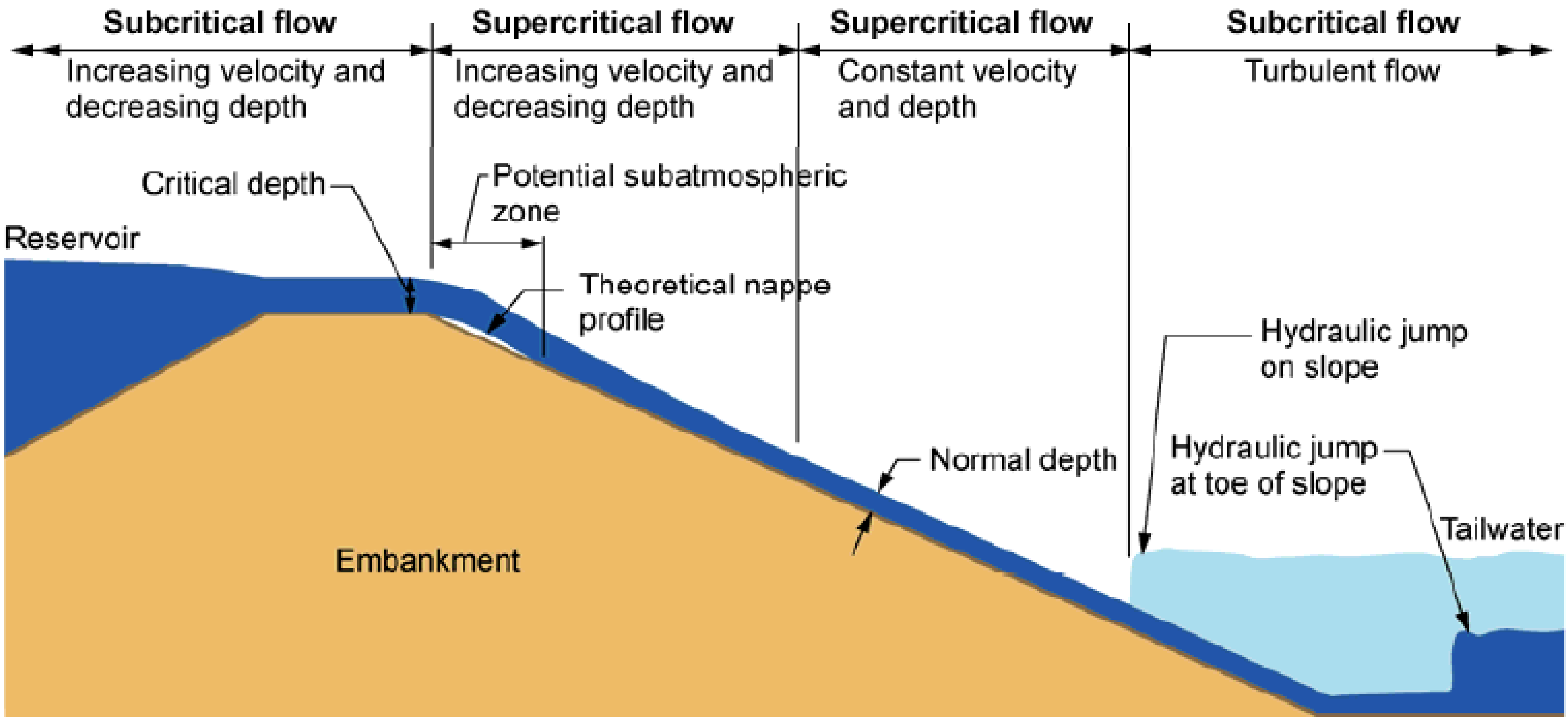


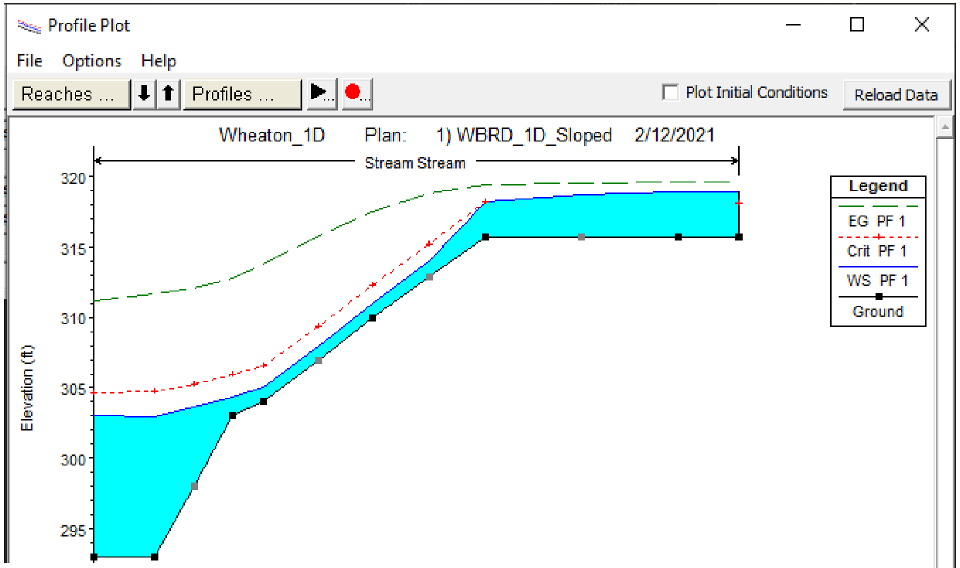
Figure 1-1.—Typical hydraulic conditions during embankment overtopping (Reclamation).

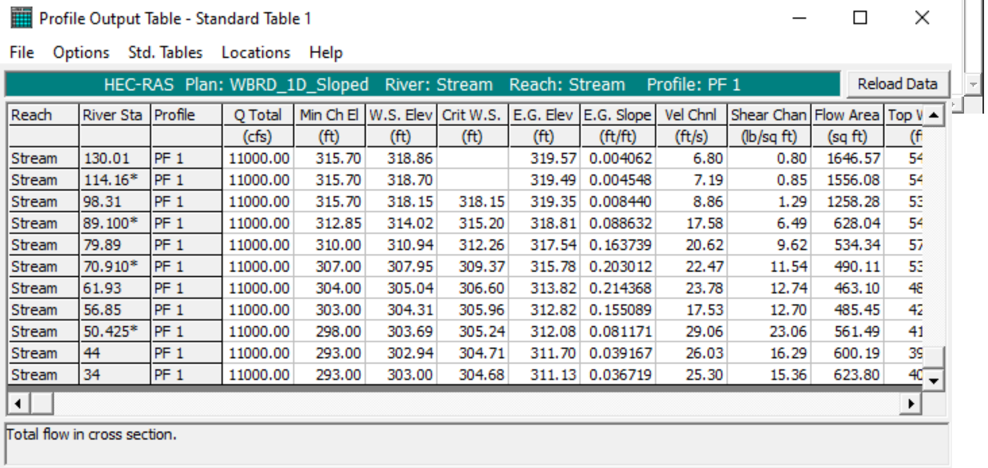
Important Flow Characteristics to Determine

- Flow Depth
- Flow Velocity
- Shear Stress
- Flow Type
- EGL
- Hydraulic jump location

Selecting a Hydraulic Model

- Selecting Hydraulic Model
  - 1-D
  - 2-D
  - 3-D

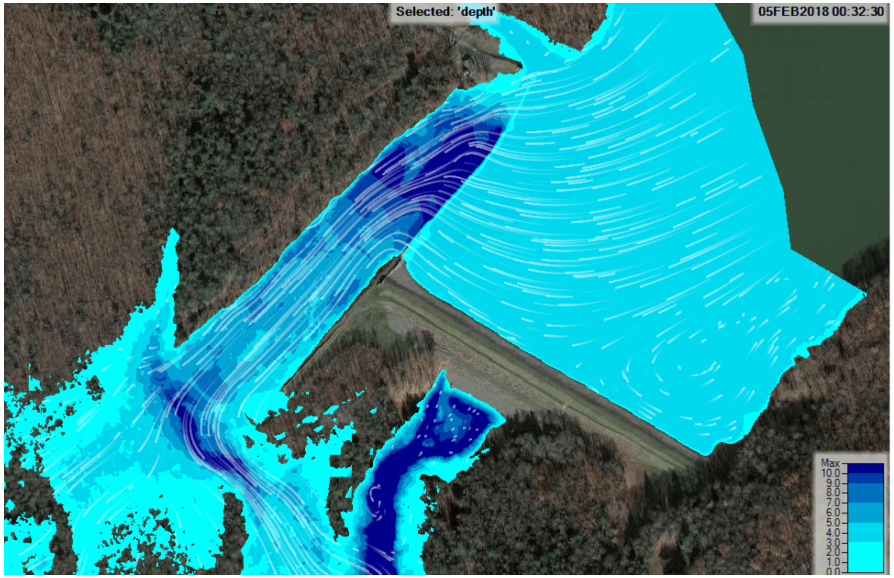




HEC-RAS Plan: WBRD_1D_Sloped River: Stream Reach: Stream Profile: PF 1

Reach	River Sta	Profile	Q Total (cfs)	Min Ch El (ft)	W.S. Elev (ft)	Crit W.S. (ft)	E.G. Elev (ft)	E.G. Slope (ft/ft)	Vel Chnl (ft/s)	Shear Chan (lb/sq ft)	Flow Area (sq ft)	Top V (f
Stream	130.01	PF 1	11000.00	315.70	318.86		319.57	0.004062	6.80	0.80	1646.57	54
Stream	114.16*	PF 1	11000.00	315.70	318.70		319.49	0.004548	7.19	0.85	1556.08	54
Stream	98.31	PF 1	11000.00	315.70	318.15	318.15	319.35	0.008440	8.86	1.29	1258.28	53
Stream	89.100*	PF 1	11000.00	312.85	314.02	315.20	318.81	0.088632	17.58	6.49	628.04	54
Stream	79.89	PF 1	11000.00	310.00	310.94	312.26	317.54	0.163739	20.62	9.62	534.34	57
Stream	70.910*	PF 1	11000.00	307.00	307.95	309.37	315.78	0.203012	22.47	11.54	490.11	53
Stream	61.93	PF 1	11000.00	304.00	305.04	306.60	313.82	0.214368	23.78	12.74	463.10	48
Stream	56.85	PF 1	11000.00	303.00	304.31	305.96	312.82	0.155089	17.53	12.70	485.45	42
Stream	50.425*	PF 1	11000.00	298.00	303.69	305.24	312.08	0.081171	29.06	23.06	561.49	41
Stream	44	PF 1	11000.00	293.00	302.94	304.71	311.70	0.039167	26.03	16.29	600.19	39
Stream	34	PF 1	11000.00	293.00	303.00	304.68	311.13	0.036719	25.30	15.36	623.80	40

Total flow in cross section.



Non-flow Considerations

- Geology
- Slope Stability
- Seepage
- Dam “Guts”

Other Considerations

General design considerations when selecting an overtopping protection system for a particular project may include:

- Unit discharge
- Maximum head on crest
- Embankment or drop height
- Embankment materials
- Downstream slope flow duration
- Flow velocity
- Shear stress
- Surface discontinuities that can lead to irregular hydraulic flow patterns or turbulence
- Potential for differential settlement
- Cavitation potential
- Erosion potential stagnation (or uplift) pressures
- Aesthetics
- Economics
- Potential for debris loads
- Durability (or resistance to corrosion abrasion and freeze-thaw damage)
- Energy dissipation
- Downstream channel conditions
- Downstream consequences
- Constructability
- Maintenance requirements
- Potential vulnerabilities (including terrorism and vandalism)
- Risks

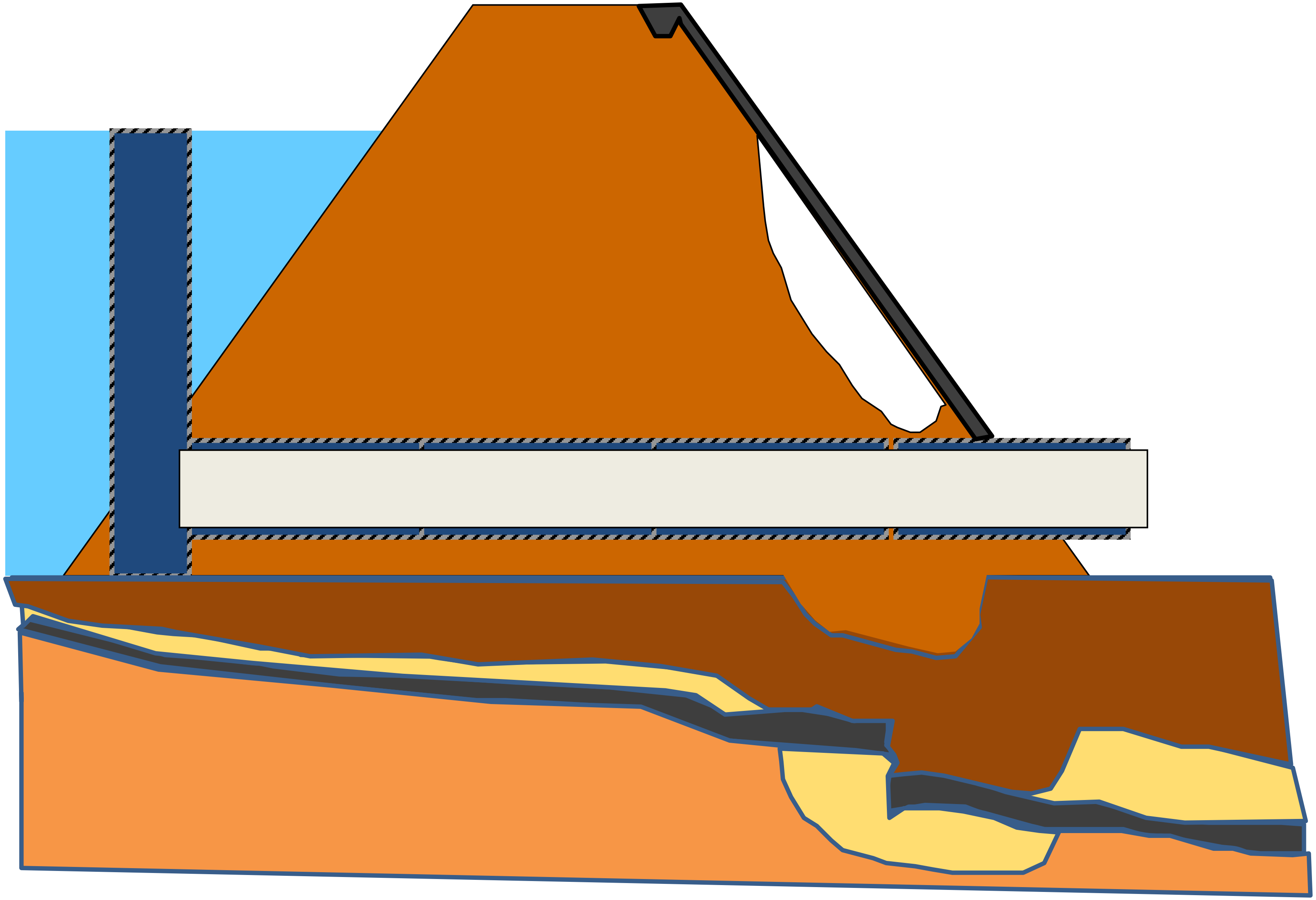
Preliminary Studies

- Goal – understand conditions of embankment, foundation, downstream areas and to develop appropriate geotechnical parameters for:
  - Analyzing embankment slope stability and seepage conditions
  - Estimating the bearing capacity of the foundation
  - Providing analysis of filter compatibility
  - Predicting settlement or heave
- Design drawings, studies, construction records, inspections, instrumentation, etc.

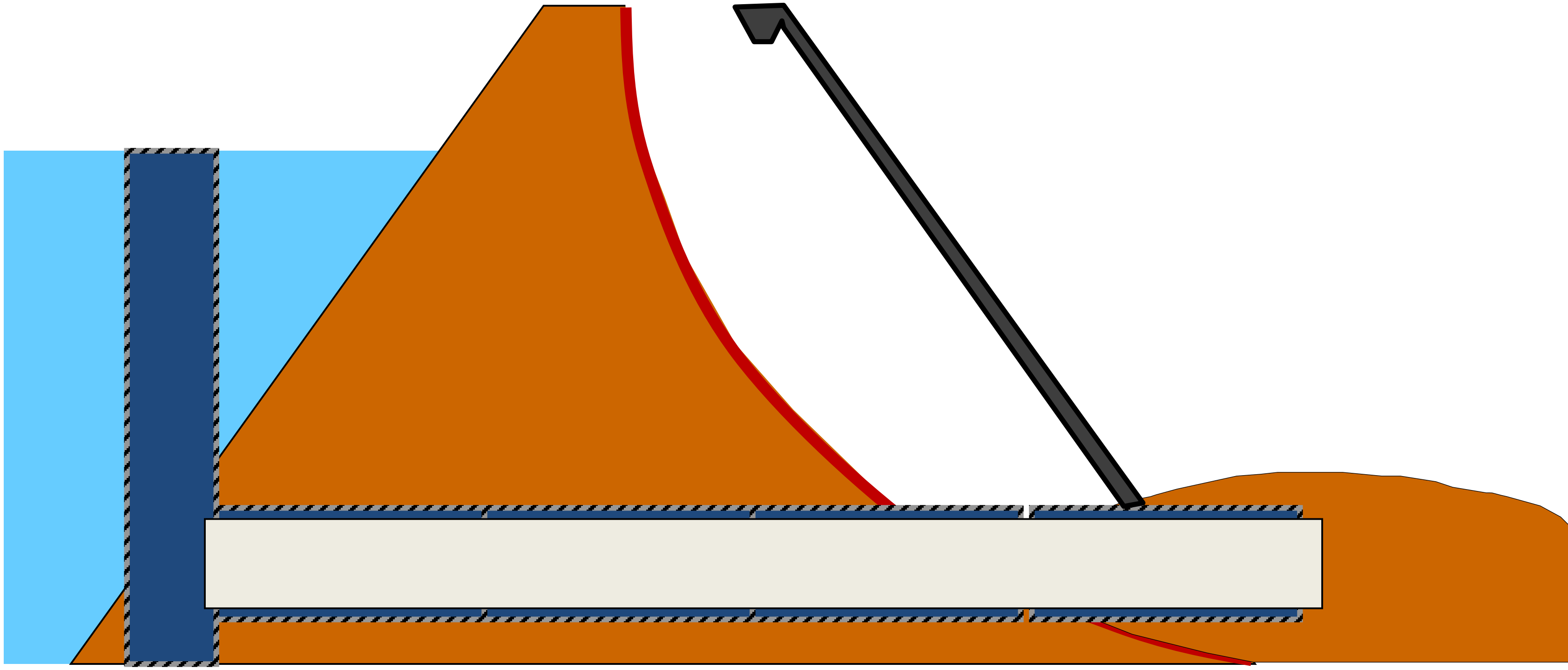
Subsurface Investigations

- Goal – determine subsurface strata and water levels in embankment and foundation, and to collect sample for lab testing
  - Drilling test holes/test pits (*requires permit)*
    - Logging and sampling
    - Classify soils encountered,
  - Geophysics, other non-destructing testing
  - Evaluate existing drain pipes
  - Identify and locate underground utilities
  - Others
    - Permeability
    - Consolidation tests
    - Direct-shear or triaxial-shear testing
    - Chemical testing
    - Dispersion tests
  - Scope will vary based on complexity of the dam embankment

Geologic Considerations

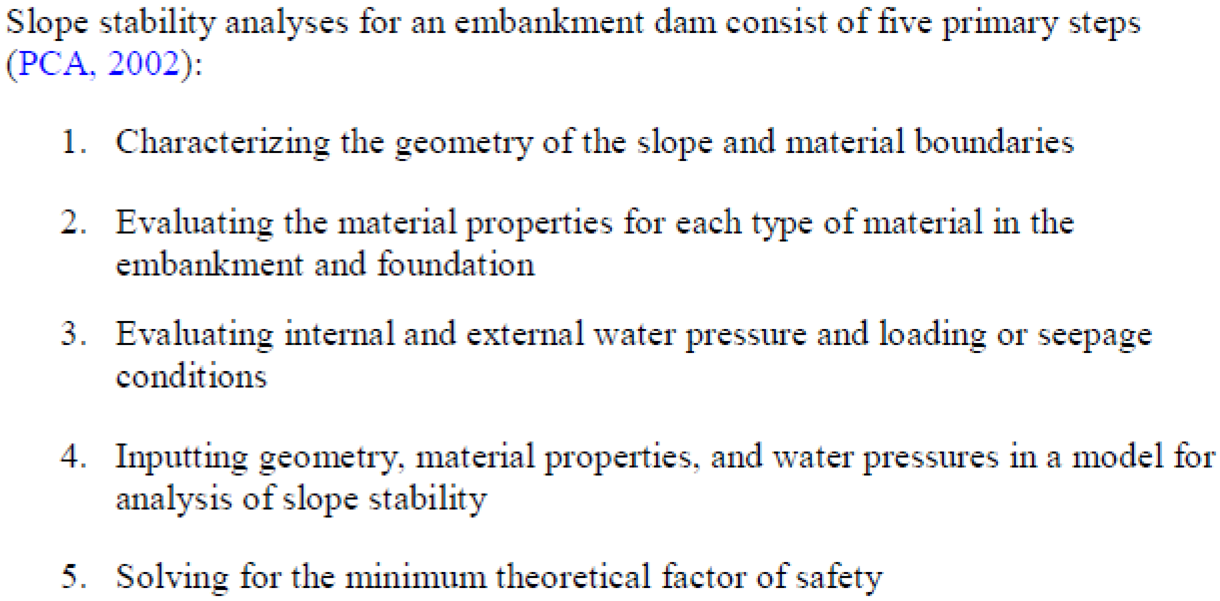


Slope Stability



Slope Stability Analyses

Stability during construction, during normal loading, during max loading conditions.

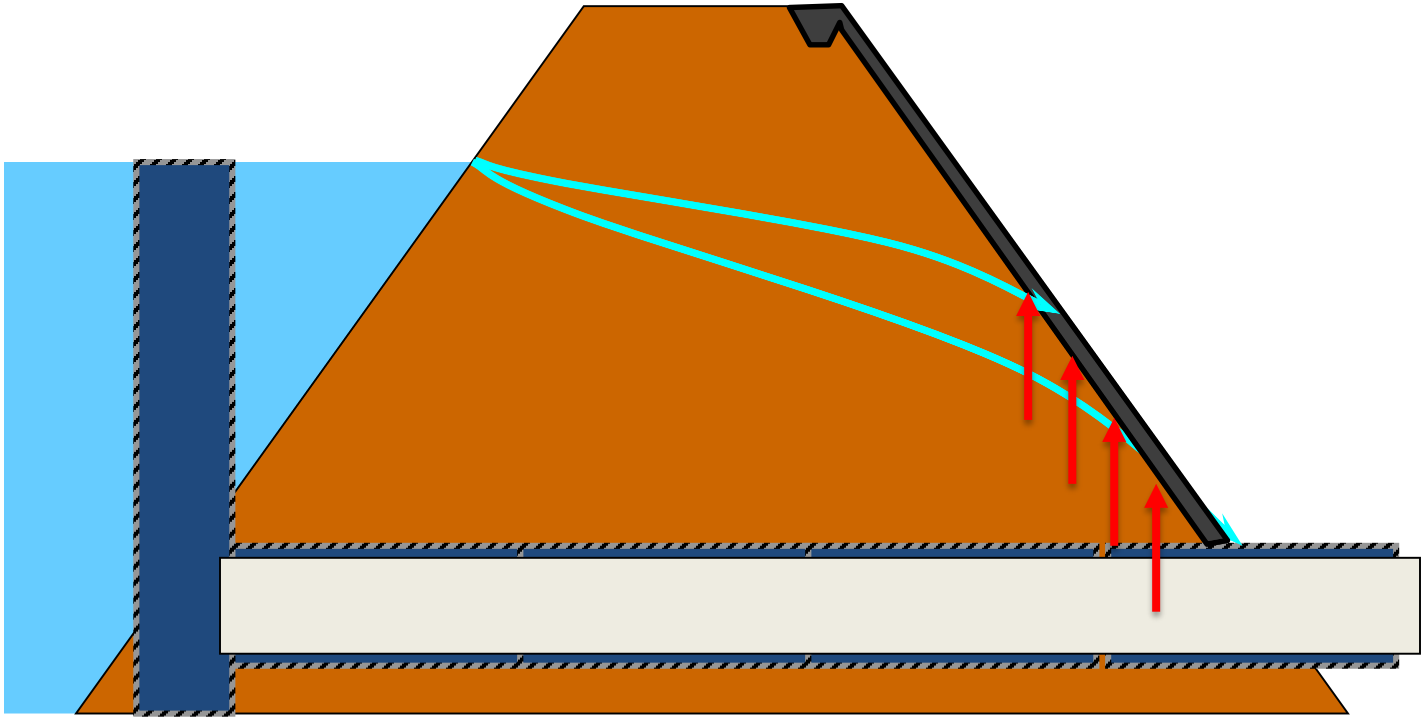


Slope stability analyses for an embankment dam consist of five primary steps (PCA, 2002):

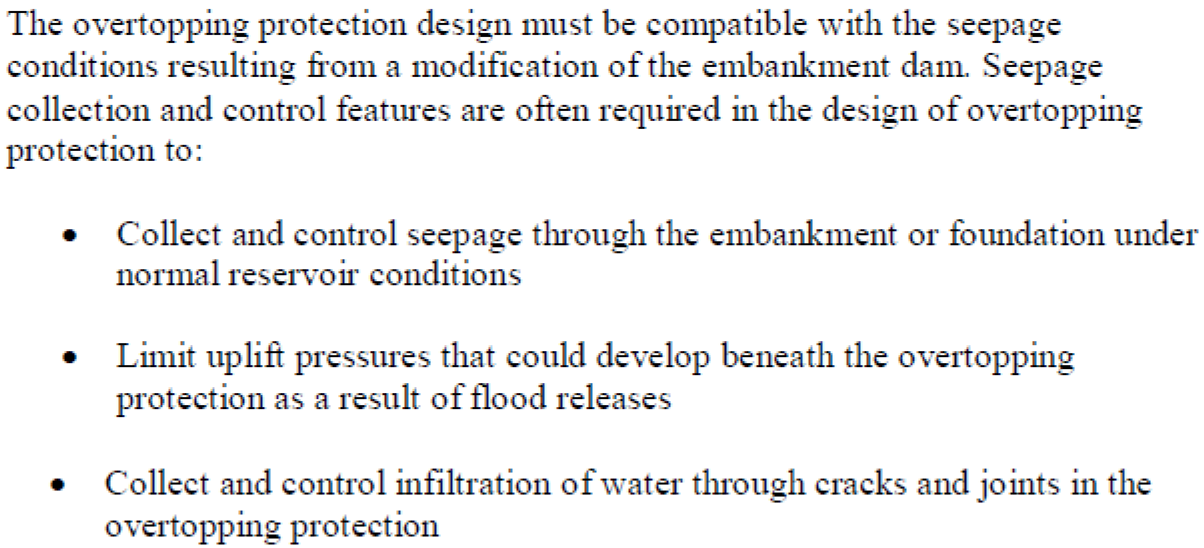
1. Characterizing the geometry of the slope and material boundaries
2. Evaluating the material properties for each type of material in the embankment and foundation
3. Evaluating internal and external water pressure and loading or seepage conditions
4. Inputting geometry, material properties, and water pressures in a model for analysis of slope stability
5. Solving for the minimum theoretical factor of safety

IMPORTANT:
Hire a qualified, licensed engineer with experience performing these analyses

Seepage



Seepage Analysis

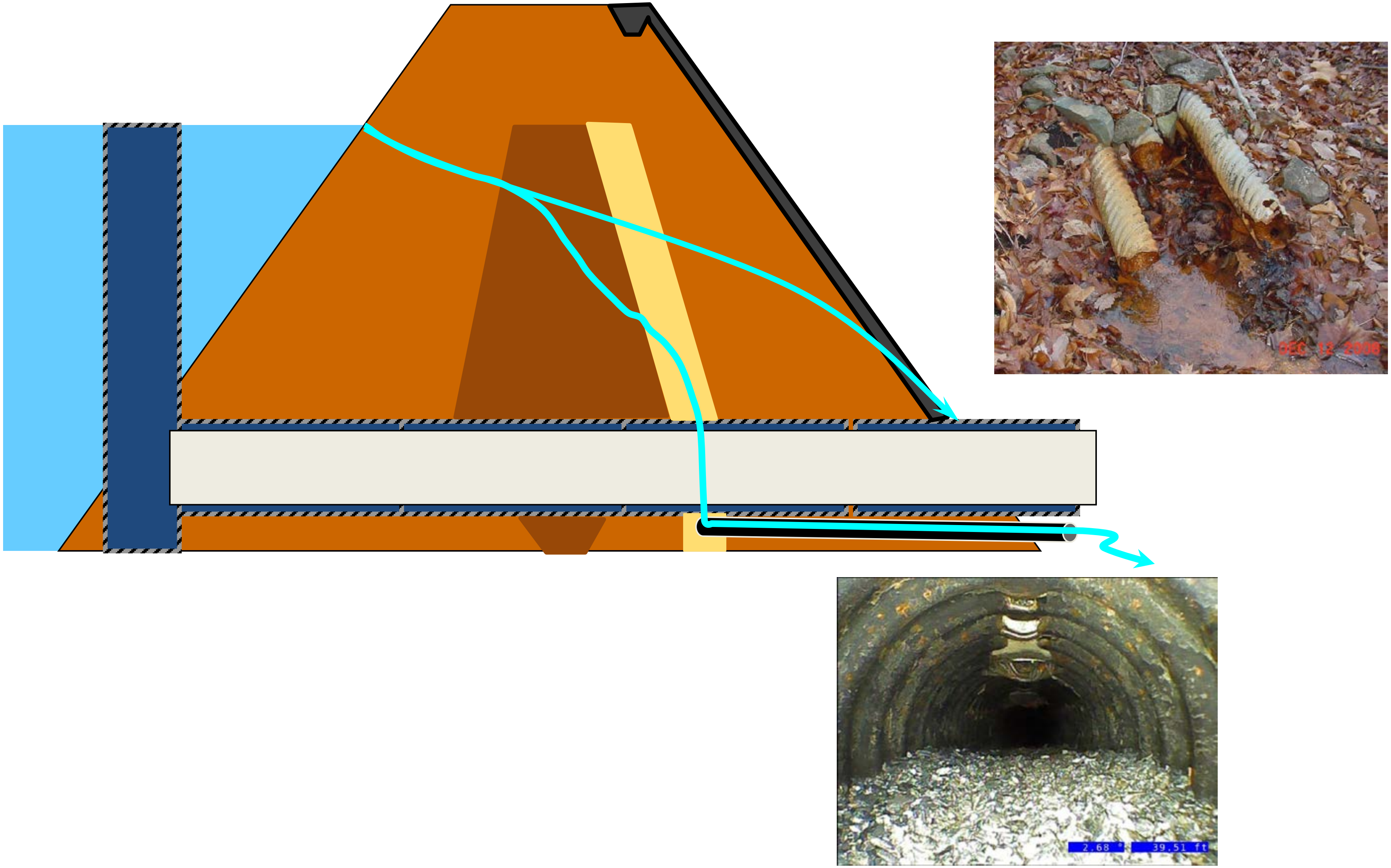


The overtopping protection design must be compatible with the seepage conditions resulting from a modification of the embankment dam. Seepage collection and control features are often required in the design of overtopping protection to:

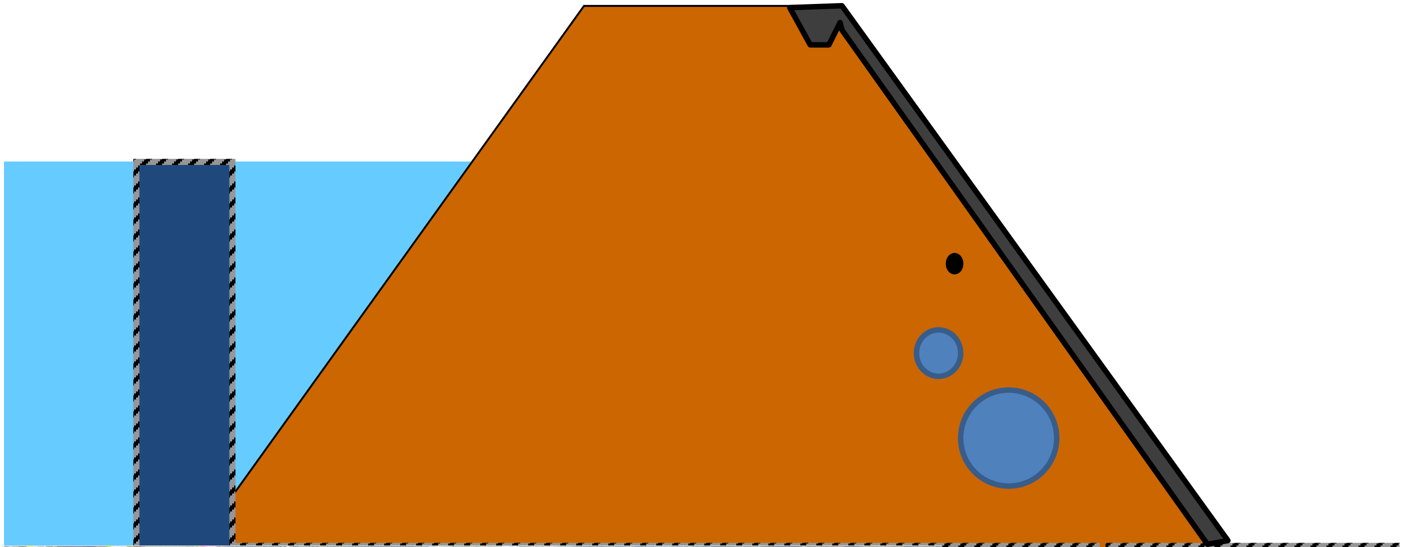
- Collect and control seepage through the embankment or foundation under normal reservoir conditions
- Limit uplift pressures that could develop beneath the overtopping protection as a result of flood releases
- Collect and control infiltration of water through cracks and joints in the overtopping protection

Principal Spillway Condition

Filter Diaphragm/Drains



Utilities



Roller-Compacted Concrete

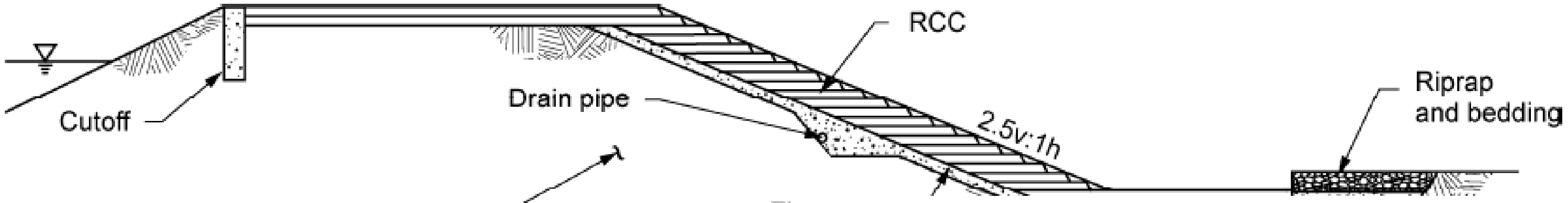


Figure 2-3.—Typical section:
(PCA, 2002, all ri

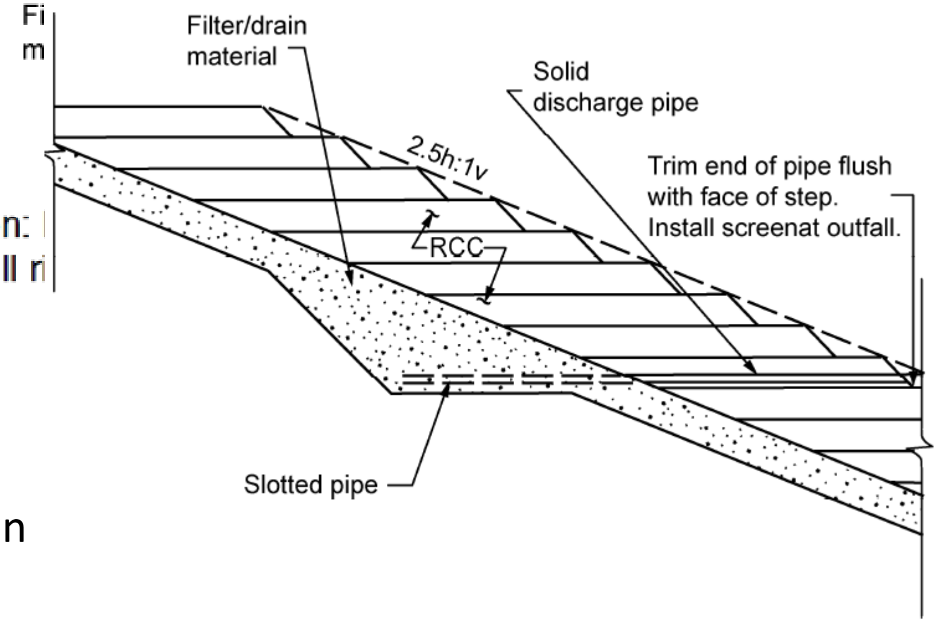


Figure 2-4.—Typical drainage details
(PCA, 2002, all rights reserved).

- Highest flow tolerance
- Non-proprietary
- Relative ease of design and construction

Design Guidance

- PCA, Design Manual for RCC Spillways and Overtopping Protection, 2002

Concrete Spillway

- Cracking
- Settlement
- Joint Deficiencies
- Adequate Underdrains

Concrete Spillways – Joints and Cracks

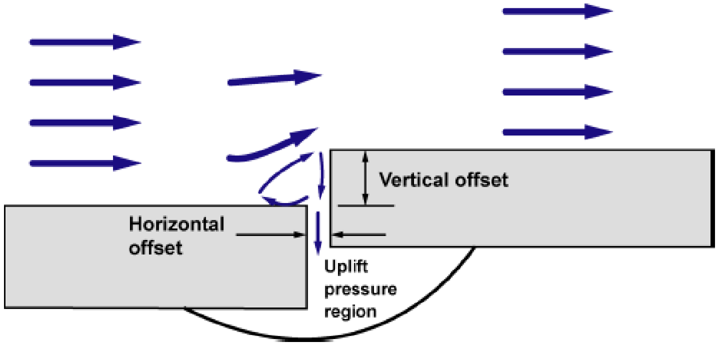


Figure 3-1.—Development of spillway stagnation pressures (Reclamation).

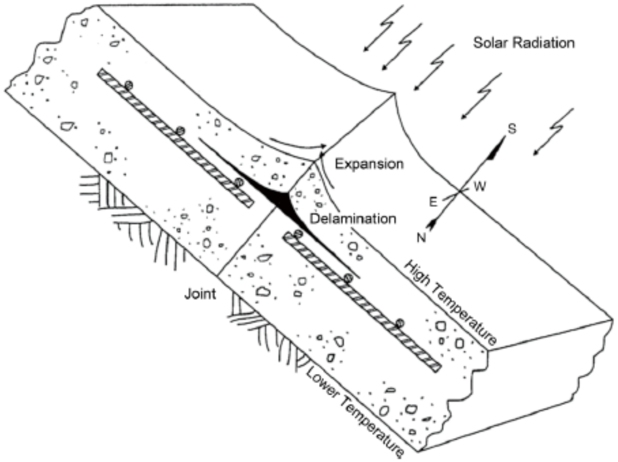
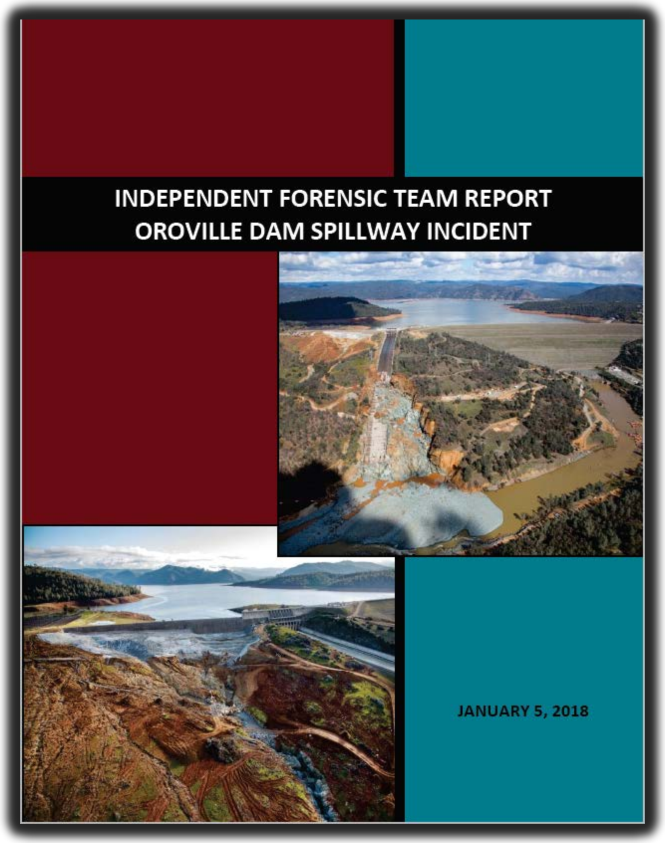


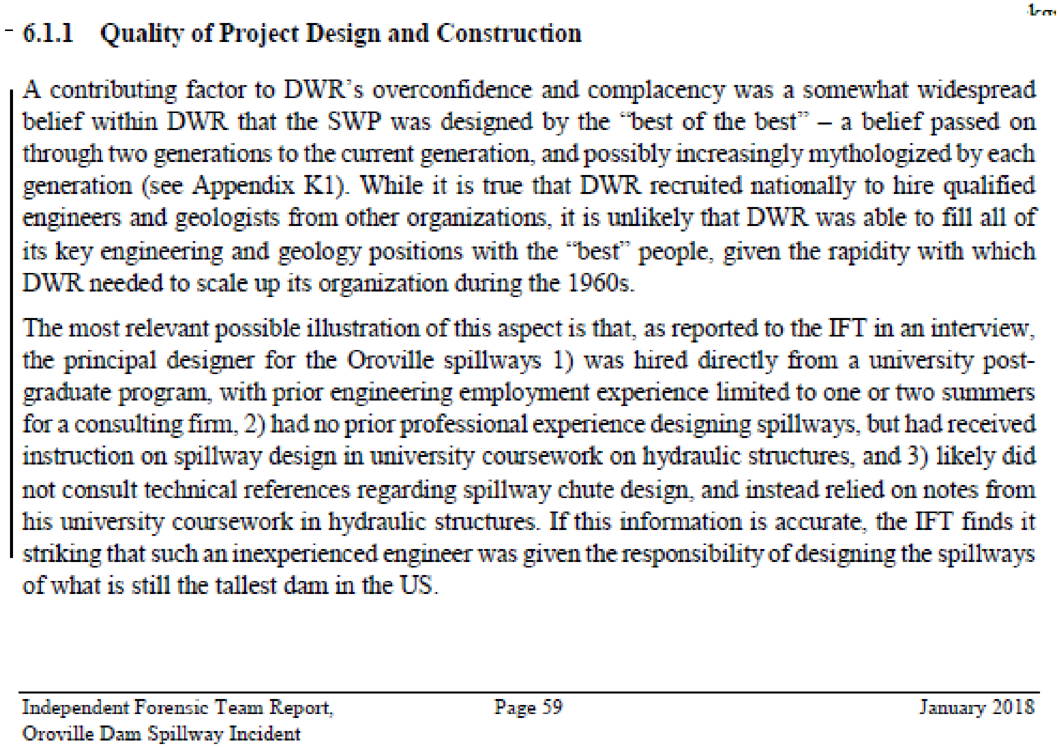
Figure 3-2.—Concrete delamination due to thermal expansion (Reclamation, 1997).

Oroville Example



INDEPENDENT FORENSIC TEAM REPORT
OROVILLE DAM SPILLWAY INCIDENT

JANUARY 5, 2018



6.1.1 Quality of Project Design and Construction

A contributing factor to DWR's overconfidence and complacency was a somewhat widespread belief within DWR that the SWP was designed by the "best of the best" – a belief passed on through two generations to the current generation, and possibly increasingly mythologized by each generation (see Appendix K1). While it is true that DWR recruited nationally to hire qualified engineers and geologists from other organizations, it is unlikely that DWR was able to fill all of its key engineering and geology positions with the "best" people, given the rapidity with which DWR needed to scale up its organization during the 1960s.

The most relevant possible illustration of this aspect is that, as reported to the IFT in an interview, the principal designer for the Oroville spillways 1) was hired directly from a university post-graduate program, with prior engineering employment experience limited to one or two summers for a consulting firm, 2) had no prior professional experience designing spillways, but had received instruction on spillway design in university coursework on hydraulic structures, and 3) likely did not consult technical references regarding spillway chute design, and instead relied on notes from his university coursework in hydraulic structures. If this information is accurate, the IFT finds it striking that such an inexperienced engineer was given the responsibility of designing the spillways of what is still the tallest dam in the US.

Independent Forensic Team Report, Oroville Dam Spillway Incident — Page 59 — January 2018

https://water.ca.gov/Programs/State-Water-Project/SWP-Facilities/Oroville/Oroville-Spillways/Forensic-Team

Concrete Spillway Joints

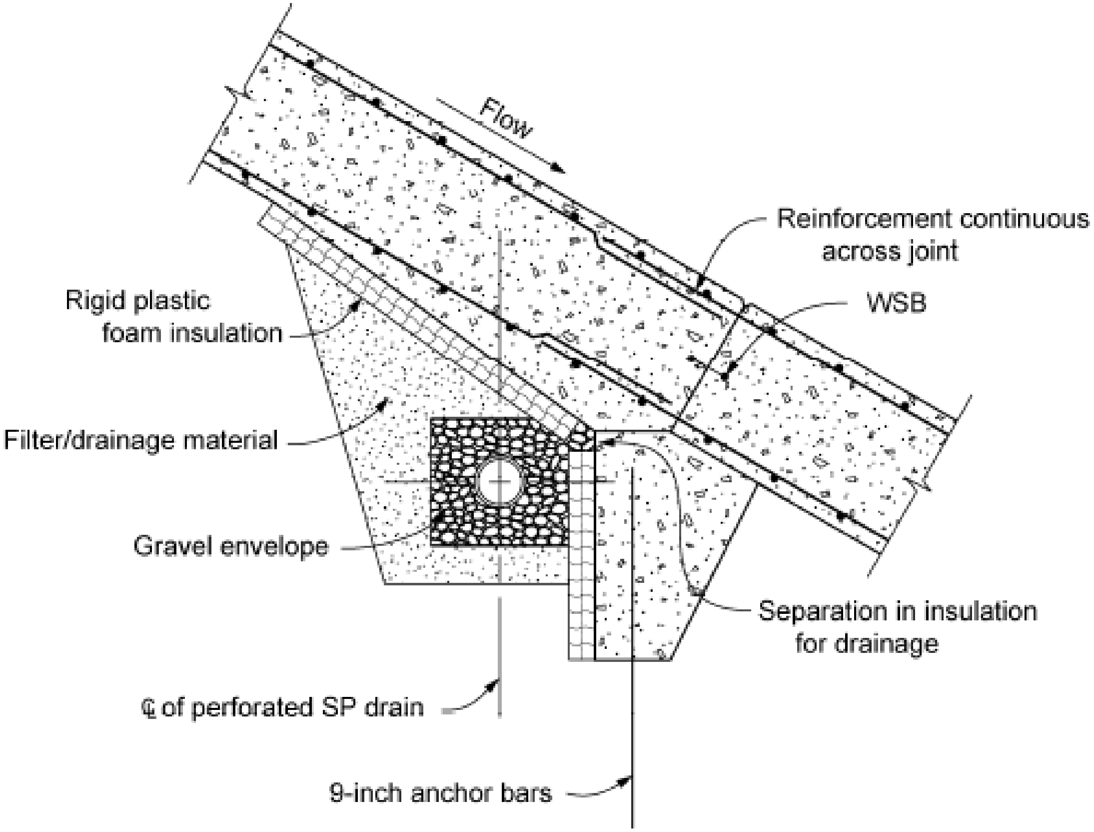


Figure 3-8.—Defensive design measures for concrete chutes to prevent uplift failure (Reclamation, courtesy of Bill Fiedler).

Concrete Spillways - Cavitation

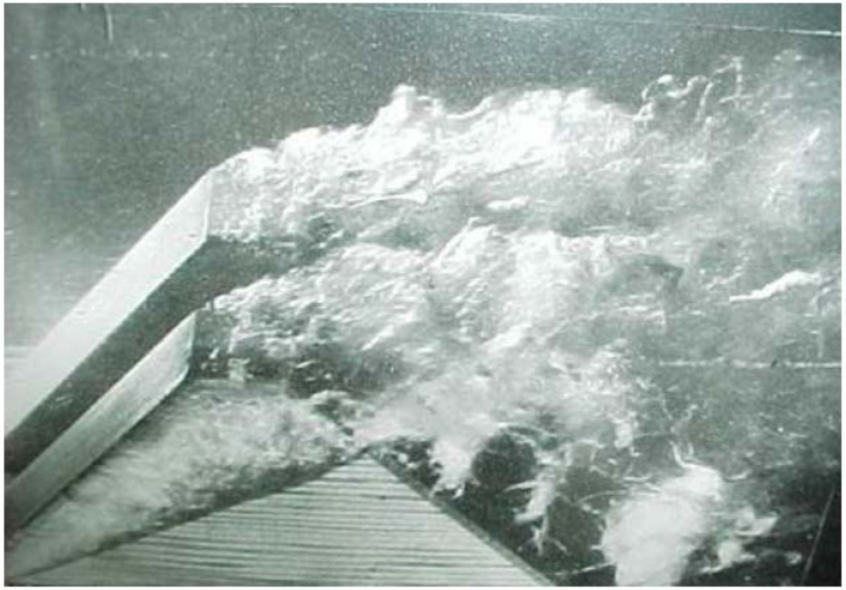
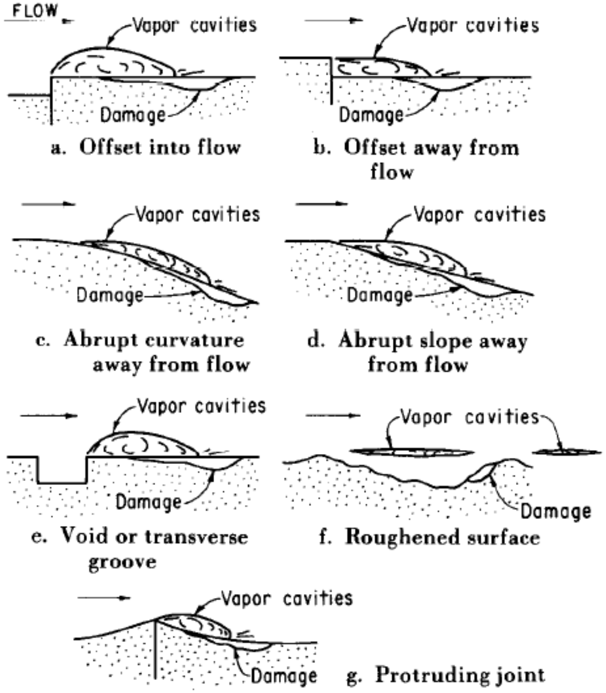


Figure 3-9.—Cavitation created in low ambient pressure chamber (Reclamation, 1990a).



a. Offset into flow

b. Offset away from flow

c. Abrupt curvature away from flow

d. Abrupt slope away from flow

e. Void or transverse groove

f. Roughened surface

g. Protruding joint

Figure 2-1.—*Typical isolated roughness elements found in hydraulic structures.*

Precast Concrete Blocks

- – Cable-tied
- – Interlocking
- – Overlapping
- – Butt-jointed

*Proprietary

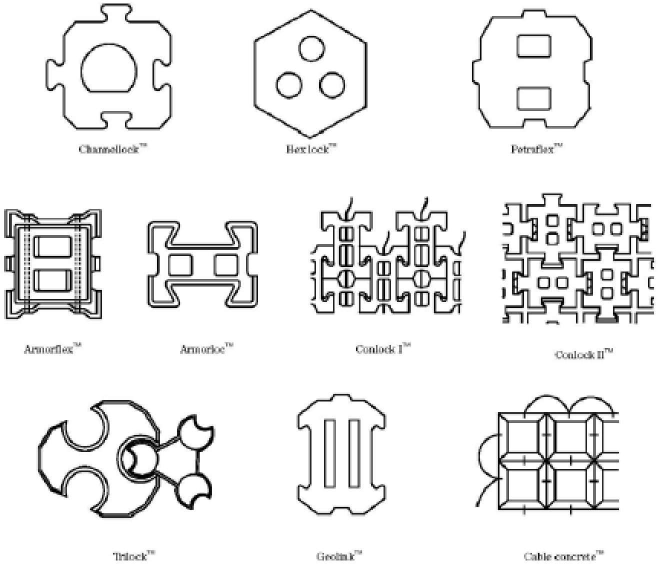
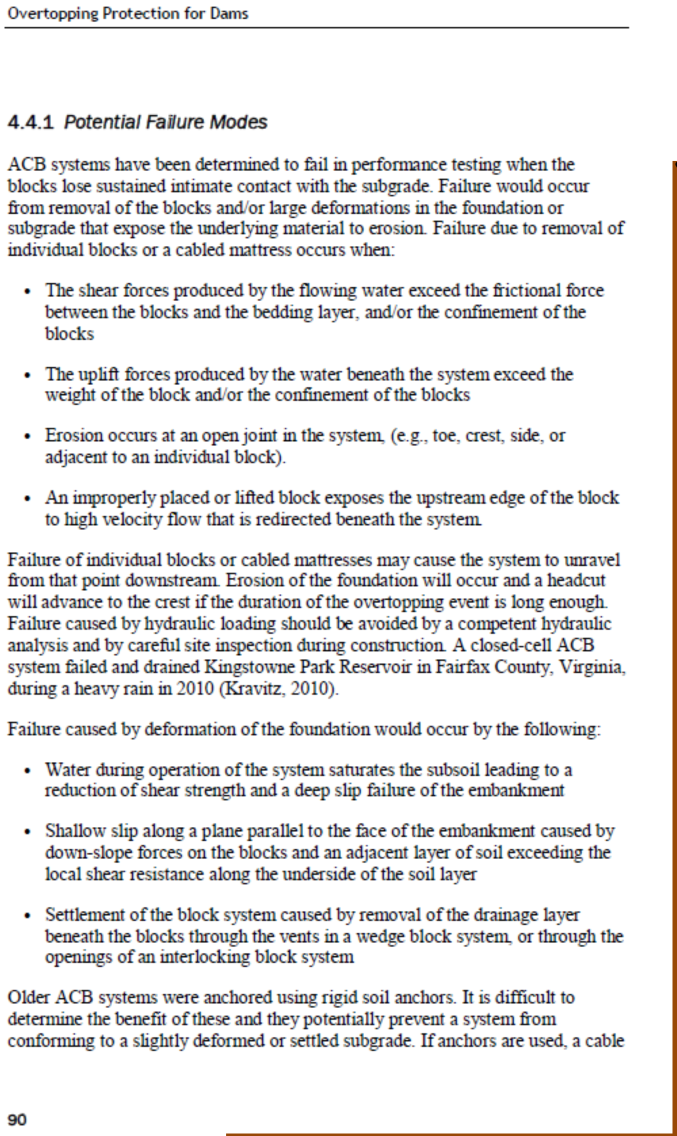


Figure 4-2.—Common examples of precast concrete revetment systems. (NRCS, 2007).

<u>Design Guidance</u>:

- National Engineering Handbook Chapter 54, Articulated Concrete Block Armored Spillways
- National Concrete Masonry Association (NCMA), Design Manual for ACB Revetment Systems

General Configuration



Overtopping Protection for Dams

4.4.1 *Potential Failure Modes*

ACB systems have been determined to fail in performance testing when the blocks lose sustained intimate contact with the subgrade. Failure would occur from removal of the blocks and/or large deformations in the foundation or subgrade that expose the underlying material to erosion. Failure due to removal of individual blocks or a cabled mattress occurs when:

- The shear forces produced by the flowing water exceed the frictional force between the blocks and the bedding layer, and/or the confinement of the blocks
- The uplift forces produced by the water beneath the system exceed the weight of the block and/or the confinement of the blocks
- Erosion occurs at an open joint in the system, (e.g., toe, crest, side, or adjacent to an individual block).
- An improperly placed or lifted block exposes the upstream edge of the block to high velocity flow that is redirected beneath the system.

Failure of individual blocks or cabled mattresses may cause the system to unravel from that point downstream. Erosion of the foundation will occur and a headcut will advance to the crest if the duration of the overtopping event is long enough. Failure caused by hydraulic loading should be avoided by a competent hydraulic analysis and by careful site inspection during construction. A closed-cell ACB system failed and drained Kingstowne Park Reservoir in Fairfax County, Virginia, during a heavy rain in 2010 (Kravitz, 2010).

Failure caused by deformation of the foundation would occur by the following:

- Water during operation of the system saturates the subsoil leading to a reduction of shear strength and a deep slip failure of the embankment
- Shallow slip along a plane parallel to the face of the embankment caused by down-slope forces on the blocks and an adjacent layer of soil exceeding the local shear resistance along the underside of the soil layer
- Settlement of the block system caused by removal of the drainage layer beneath the blocks through the vents in a wedge block system, or through the openings of an interlocking block system

Older ACB systems were anchored using rigid soil anchors. It is difficult to determine the benefit of these and they potentially prevent a system from conforming to a slightly deformed or settled subgrade. If anchors are used, a cable

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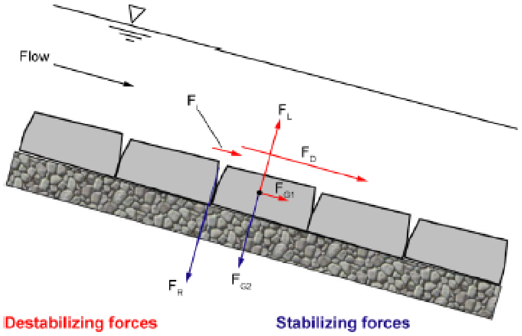
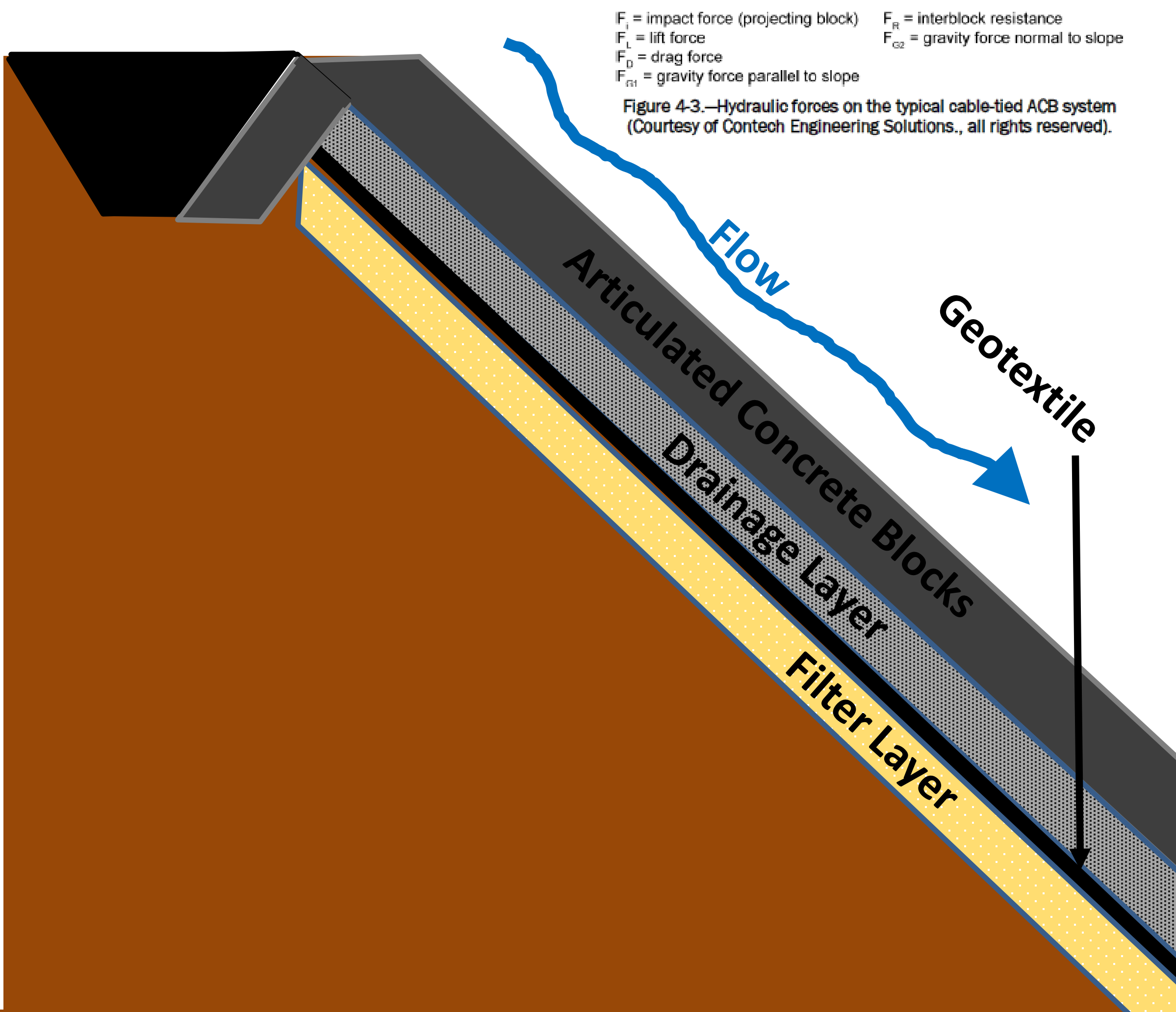


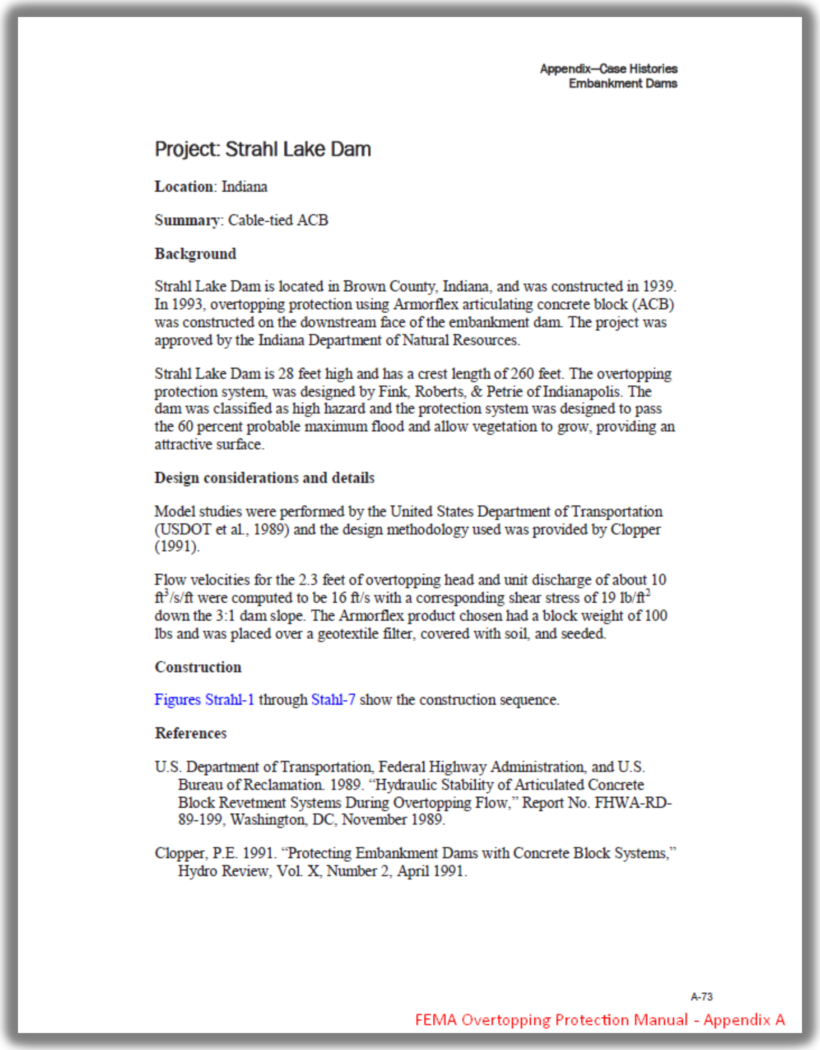
F_I = impact force (projecting block)
F_L = lift force
F_D = drag force
F_{G1} = gravity force parallel to slope

F_R = interblock resistance
F_{G2} = gravity force normal to slope

Figure 4-3.—Hydraulic forces on the typical cable-tied ACB system (Courtesy of Contech Engineering Solutions., all rights reserved).



Example



Project: Strahl Lake Dam

Location: Indiana

Summary: Cable-tied ACB

Background

Strahl Lake Dam is located in Brown County, Indiana, and was constructed in 1939. In 1993, overtopping protection using Armorflex articulating concrete block (ACB) was constructed on the downstream face of the embankment dam. The project was approved by the Indiana Department of Natural Resources.

Strahl Lake Dam is 28 feet high and has a crest length of 260 feet. The overtopping protection system, was designed by Fink, Roberts, & Petrie of Indianapolis. The dam was classified as high hazard and the protection system was designed to pass the 60 percent probable maximum flood and allow vegetation to grow, providing an attractive surface.

Design considerations and details

Model studies were performed by the United States Department of Transportation (USDOT et al., 1989) and the design methodology used was provided by Clopper (1991).

Flow velocities for the 2.3 feet of overtopping head and unit discharge of about 10 $ft^3/s/ft$ were computed to be 16 ft/s with a corresponding shear stress of 19 lb/ft^2 down the 3:1 dam slope. The Armorflex product chosen had a block weight of 100 lbs and was placed over a geotextile filter, covered with soil, and seeded.

Construction

Figures Strahl-1 through Stahl-7 show the construction sequence.

References

U.S. Department of Transportation, Federal Highway Administration, and U.S. Bureau of Reclamation. 1989. "Hydraulic Stability of Articulated Concrete Block Revetment Systems During Overtopping Flow," Report No. FHWA-RD-89-199, Washington, DC, November 1989.

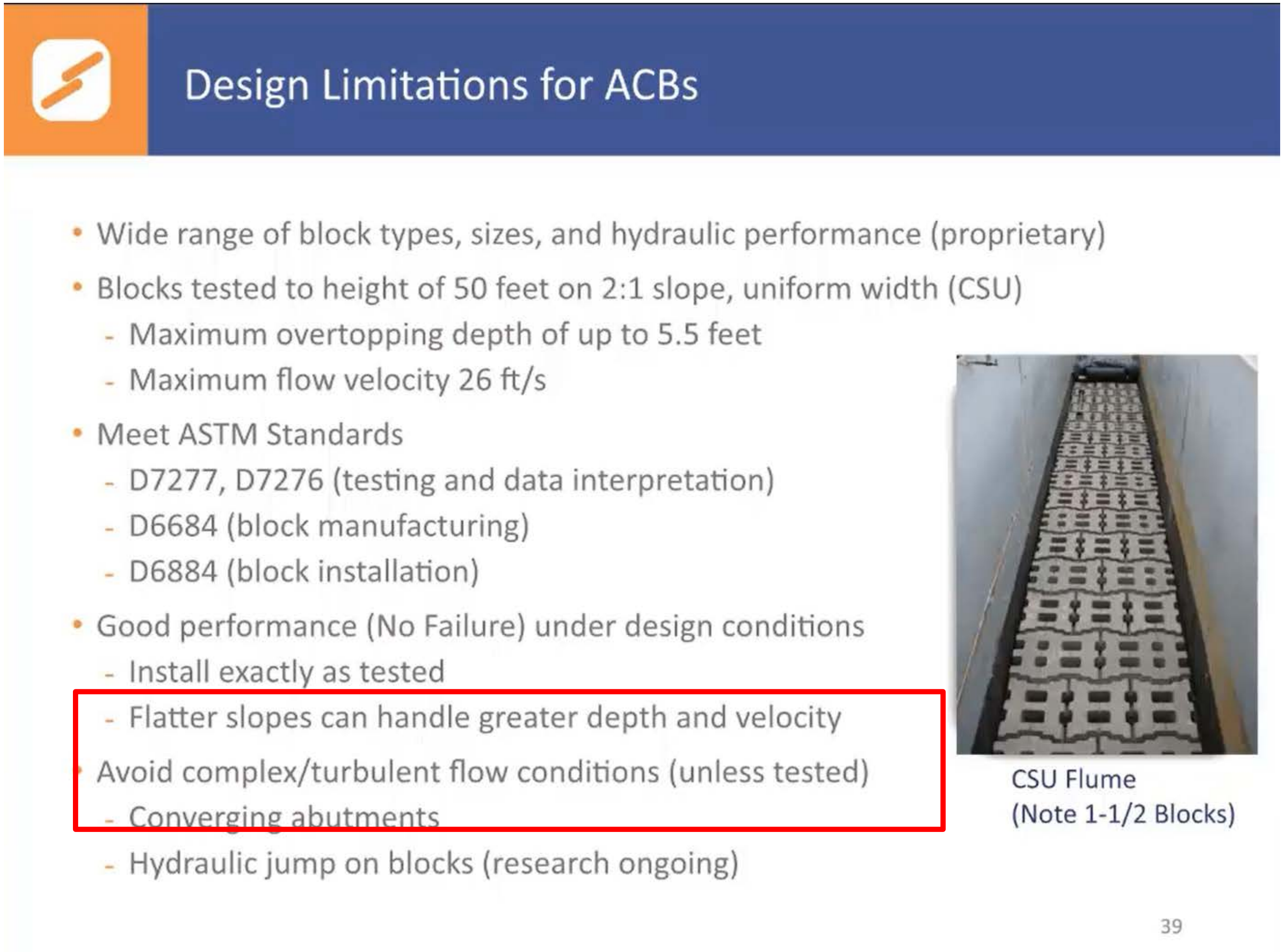
Clopper, P.E. 1991. "Protecting Embankment Dams with Concrete Block Systems," Hydro Review, Vol. X, Number 2, April 1991.

FEMA Overtopping Protection Manual - Appendix A

Figure Strahl-6.—Completed view of the cable-tied ACB system at Strahl Lake Dam for overtopping protection. Note all joints have been grouted (Courtesy of Contech Engineering Solutions, all rights reserved).

Figure Strahl-7.—Vegetative cover on Strahl Lake Dam overtopping protection (Courtesy of Contech Engineering Solutions, all rights reserved).

ACB Design Limitations



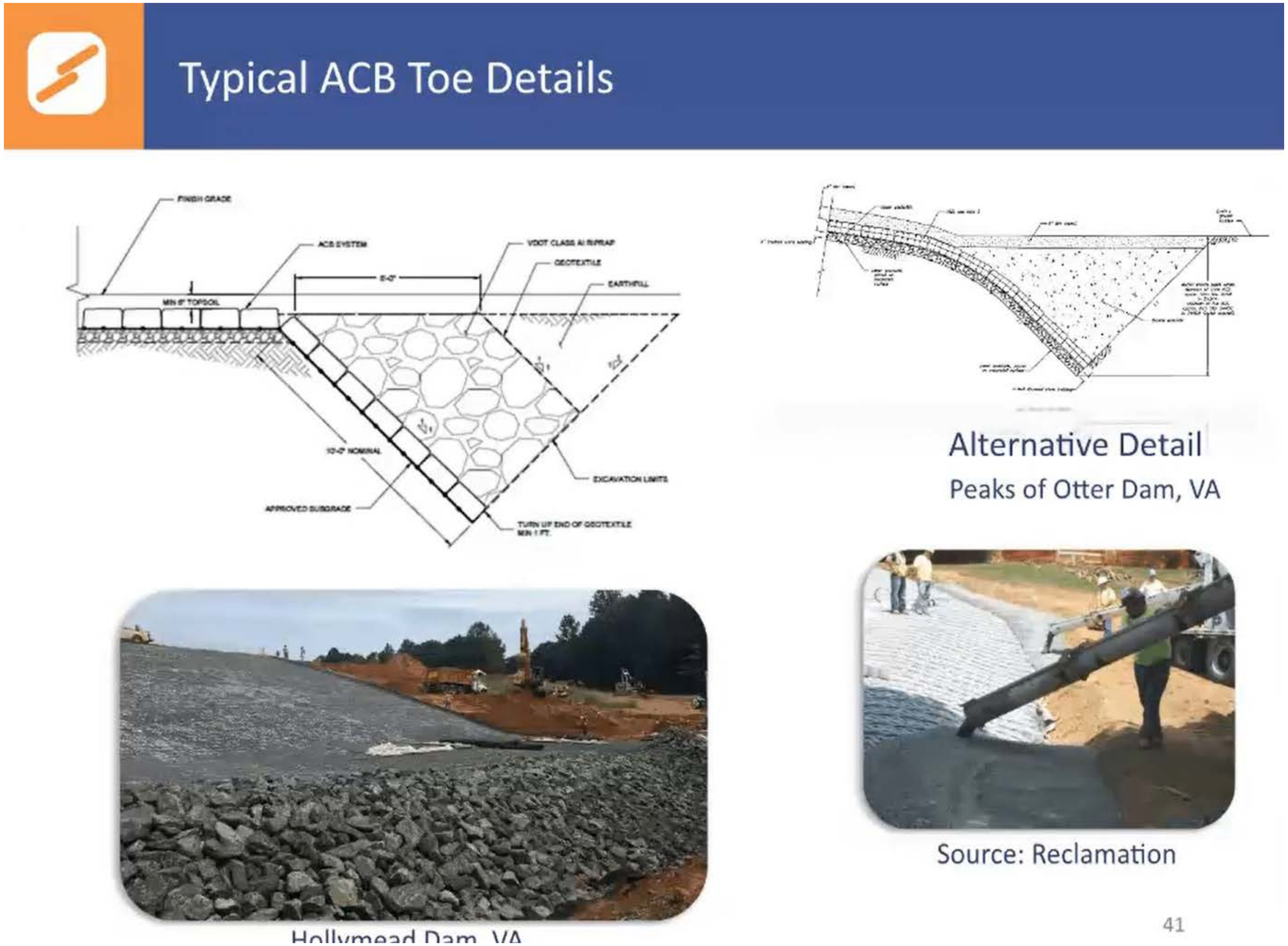
Design Limitations for ACBs

- Wide range of block types, sizes, and hydraulic performance (proprietary)
- Blocks tested to height of 50 feet on 2:1 slope, uniform width (CSU)
  - Maximum overtopping depth of up to 5.5 feet
  - Maximum flow velocity 26 ft/s
- Meet ASTM Standards
  - D7277, D7276 (testing and data interpretation)
  - D6684 (block manufacturing)
  - D6884 (block installation)
- Good performance (No Failure) under design conditions
  - Install exactly as tested
  - Flatter slopes can handle greater depth and velocity
- Avoid complex/turbulent flow conditions (unless tested)
  - Converging abutments
  - Hydraulic jump on blocks (research ongoing)

CSU Flume
(Note 1-1/2 Blocks)

Source: ASDSO Dam Overtopping Protection Systems Part 1

Concrete Block Toe Treatment



Typical ACB Toe Details

Alternative Detail

Peaks of Otter Dam, VA

Hollymead Dam, VA

Source: Reclamation

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Source: ASDSO Dam Overtopping Protection Systems Part 1

Wedge Blocks

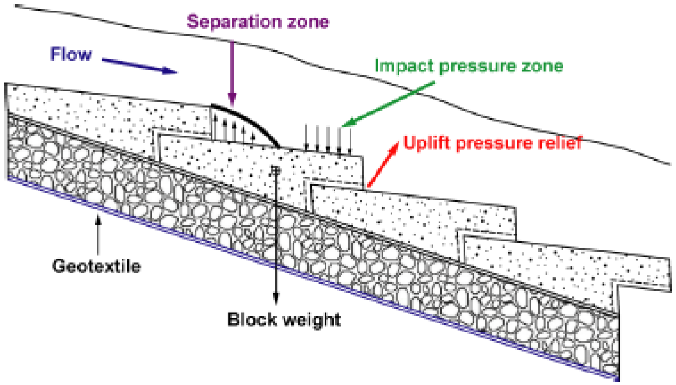


Figure 4-4.—Typical forces on a wedge-block ACB system (Courtesy of Contech Engineering Solutions, Inc., all rights reserved).

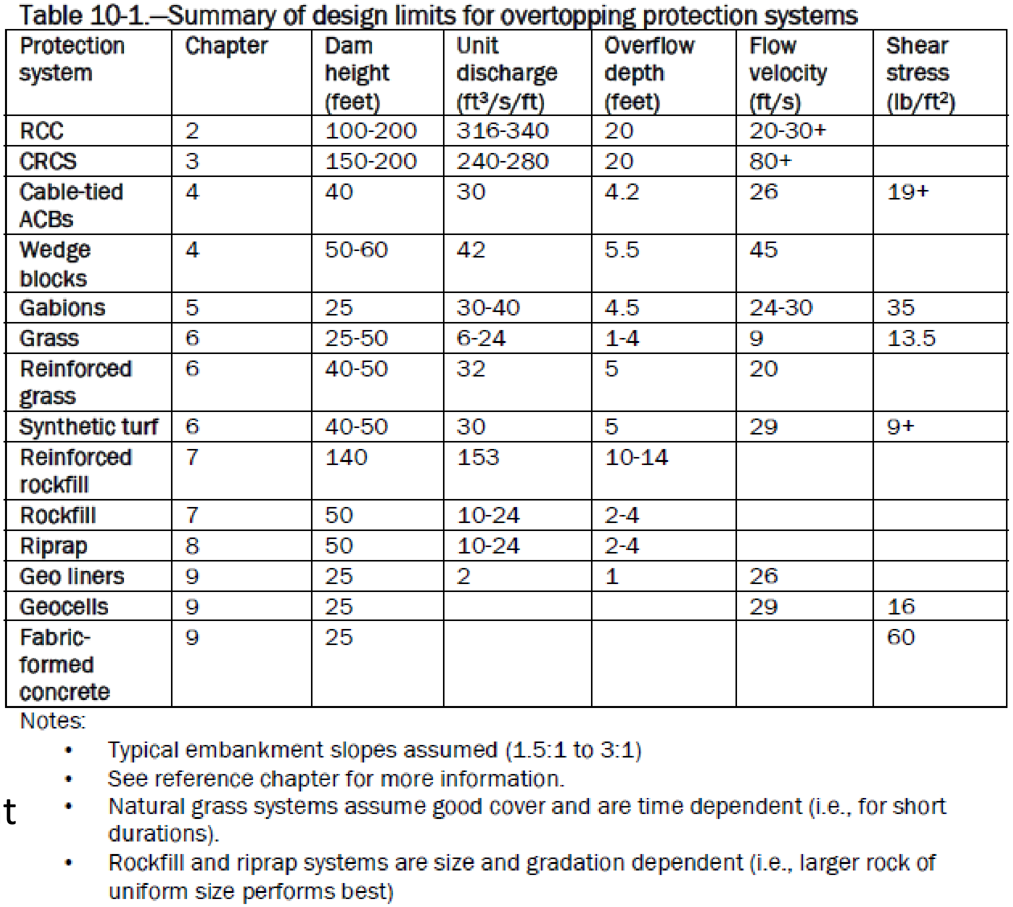


Table 10-1.—Summary of design limits for overtopping protection systems

Protection system	Chapter	Dam height (feet)	Unit discharge (ft^3/s/ft)	Overflow depth (feet)	Flow velocity (ft/s)	Shear stress (lb/ft^2)
RCC	2	100-200	316-340	20	20-30+	
CRCS	3	150-200	240-280	20	80+	
Cable-tied ACBs	4	40	30	4.2	26	19+
Wedge blocks	4	50-60	42	5.5	45	
Gabions	5	25	30-40	4.5	24-30	35
Grass	6	25-50	6-24	1-4	9	13.5
Reinforced grass	6	40-50	32	5	20	
Synthetic turf	6	40-50	30	5	29	9+
Reinforced rockfill	7	140	153	10-14		
Rockfill	7	50	10-24	2-4		
Riprap	8	50	10-24	2-4		
Geo liners	9	25	2	1	26	
Geocells	9	25			29	16
Fabric-formed concrete	9	25				60

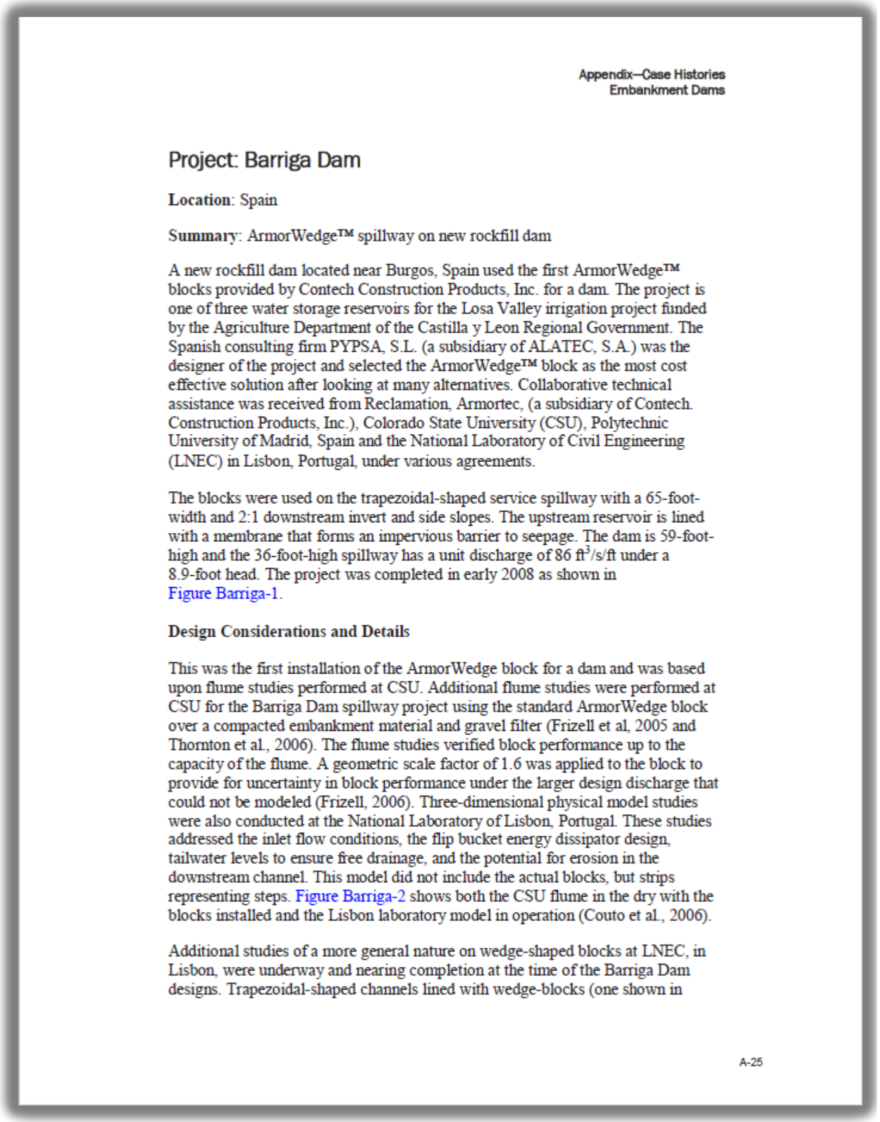
Notes:

- Typical embankment slopes assumed (1.5:1 to 3:1)
- See reference chapter for more information.
- Natural grass systems assume good cover and are time dependent (i.e., for short durations).
- Rockfill and riprap systems are size and gradation dependent (i.e., larger rock of uniform size performs best)

Design Guidance

- CIRIA, Design of Stepped Block Spillways SP-142, 1997
- FHWA, Hydraulic Stability of ACB Revetment Systems During Overtopping Flow, 1989

Example



Appendix—Case Histories
Embankment Dams

Project: Barriga Dam

Location: Spain

Summary: ArmorWedge™ spillway on new rockfill dam

A new rockfill dam located near Burgos, Spain used the first ArmorWedge™ blocks provided by Contech Construction Products, Inc. for a dam. The project is one of three water storage reservoirs for the Losa Valley irrigation project funded by the Agriculture Department of the Castilla y Leon Regional Government. The Spanish consulting firm PYPSA, S.L. (a subsidiary of ALATEC, S.A.) was the designer of the project and selected the ArmorWedge™ block as the most cost effective solution after looking at many alternatives. Collaborative technical assistance was received from Reclamation, Armortec, (a subsidiary of Contech Construction Products, Inc.), Colorado State University (CSU), Polytechnic University of Madrid, Spain and the National Laboratory of Civil Engineering (LNEC) in Lisbon, Portugal, under various agreements.

The blocks were used on the trapezoidal-shaped service spillway with a 65-foot-width and 2:1 downstream invert and side slopes. The upstream reservoir is lined with a membrane that forms an impervious barrier to seepage. The dam is 59-foot-high and the 36-foot-high spillway has a unit discharge of 86 ft^3/s/ft under a 8.9-foot head. The project was completed in early 2008 as shown in Figure Barriga-1.

Design Considerations and Details

This was the first installation of the ArmorWedge block for a dam and was based upon flume studies performed at CSU. Additional flume studies were performed at CSU for the Barriga Dam spillway project using the standard ArmorWedge block over a compacted embankment material and gravel filter (Frizell et al, 2005 and Thornton et al., 2006). The flume studies verified block performance up to the capacity of the flume. A geometric scale factor of 1.6 was applied to the block to provide for uncertainty in block performance under the larger design discharge that could not be modeled (Frizell, 2006). Three-dimensional physical model studies were also conducted at the National Laboratory of Lisbon, Portugal. These studies addressed the inlet flow conditions, the flip bucket energy dissipator design, tailwater levels to ensure free drainage, and the potential for erosion in the downstream channel. This model did not include the actual blocks, but strips representing steps. Figure Barriga-2 shows both the CSU flume in the dry with the blocks installed and the Lisbon laboratory model in operation (Couto et al., 2006).

Additional studies of a more general nature on wedge-shaped blocks at LNEC, in Lisbon, were underway and nearing completion at the time of the Barriga Dam designs. Trapezoidal-shaped channels lined with wedge-blocks (one shown in

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Appendix—Case Histories
s

Figure Barriga-14.—Completed spillway prior to operation (courtesy of Morán and Toledo, 2006, all rights reserved).

Figure Barriga-15.—Operation in May 2008 with estimated discharge of 350 – 530 ft^3/s (courtesy of Morán and Toledo, 2006, all rights reserved).

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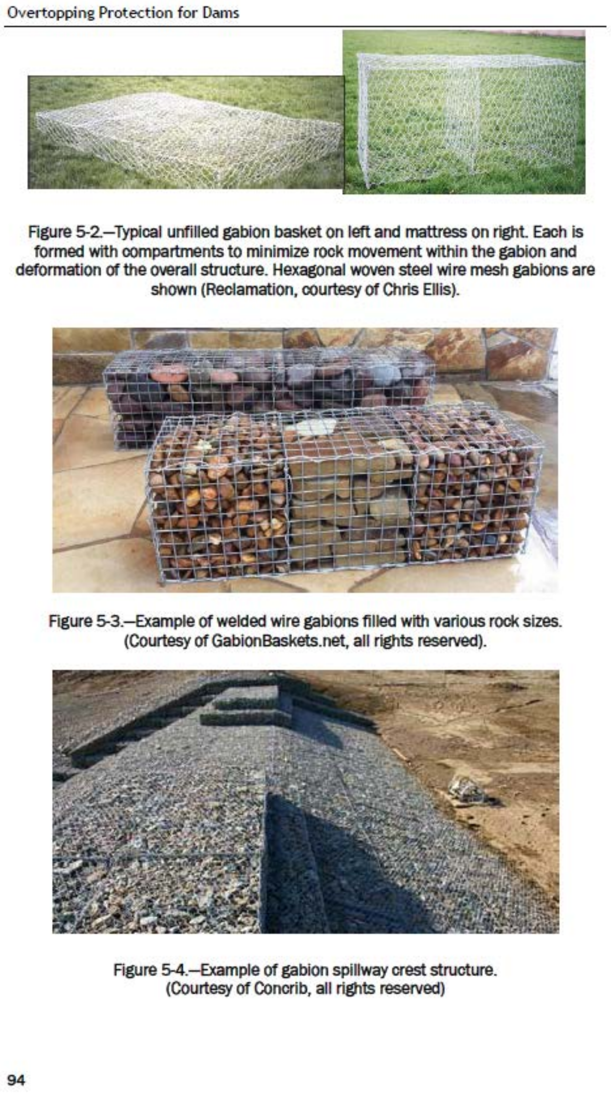
Gabions



Overtopping Protection for Dams

Figure 5-2.—Typical unfilled gabion basket on left and mattress on right. Each is formed with compartments to minimize rock movement within the gabion and deformation of the overall structure. Hexagonal woven steel wire mesh gabions are shown (Reclamation, courtesy of Chris Ellis).

Figure 5-3.—Example of welded wire gabions filled with various rock sizes. (Courtesy of GabionBaskets.net, all rights reserved).

Figure 5-4.—Example of gabion spillway crest structure. (Courtesy of Concrib, all rights reserved)

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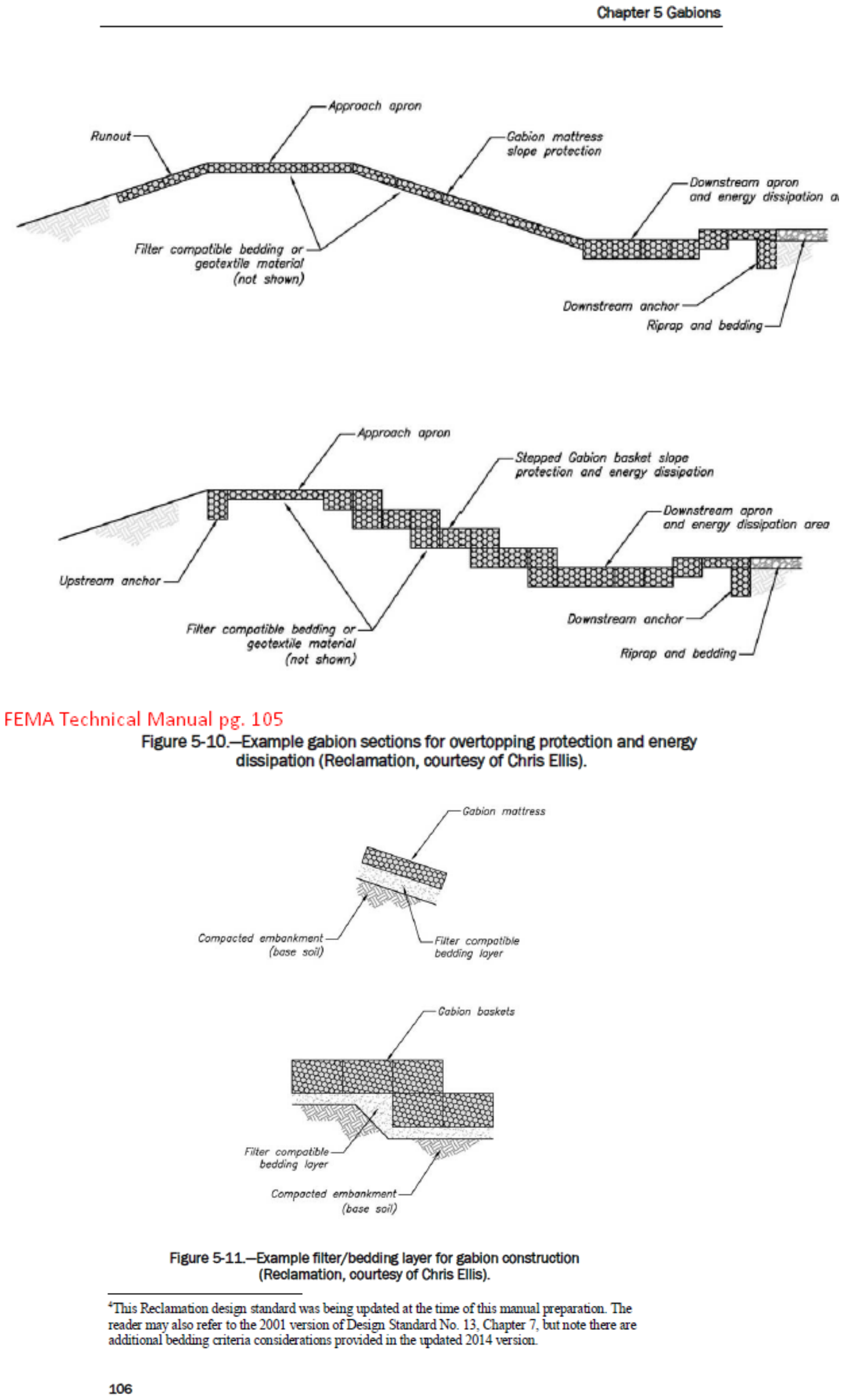

Chapter 5 Gabions

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Figure 5-10.—Example gabion sections for overtopping protection and energy dissipation (Reclamation, courtesy of Chris Ellis).

Figure 5-11.—Example filter/bedding layer for gabion construction (Reclamation, courtesy of Chris Ellis).

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[4] This Reclamation design standard was being updated at the time of this manual preparation. The reader may also refer to the 2001 version of Design Standard No. 13, Chapter 7, but note there are additional bedding criteria considerations provided in the updated 2014 version.

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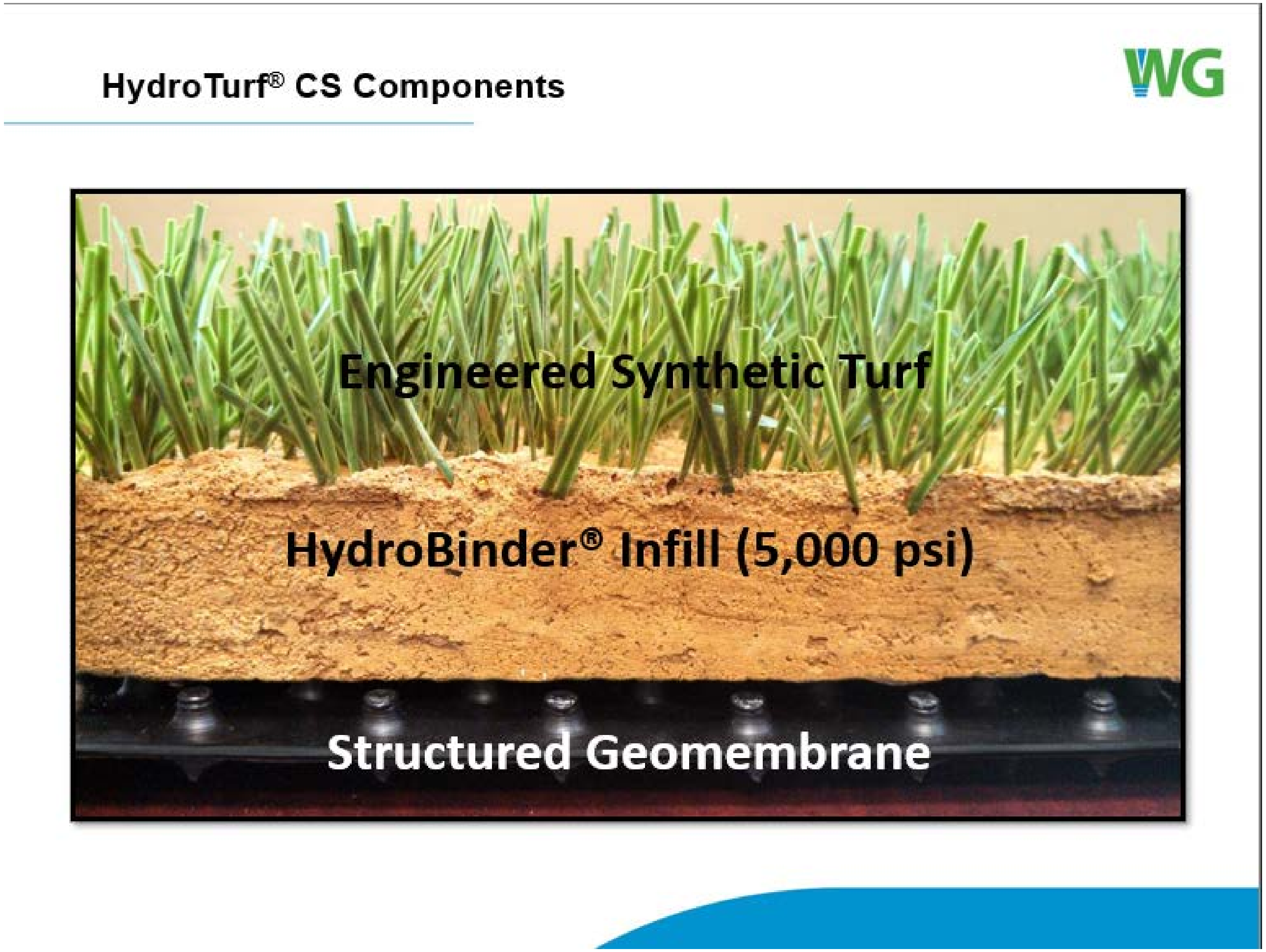
Gabions Example

Synthetic Turf Revetments - Hydroturf

Source: Watershed Geo. Used with permission

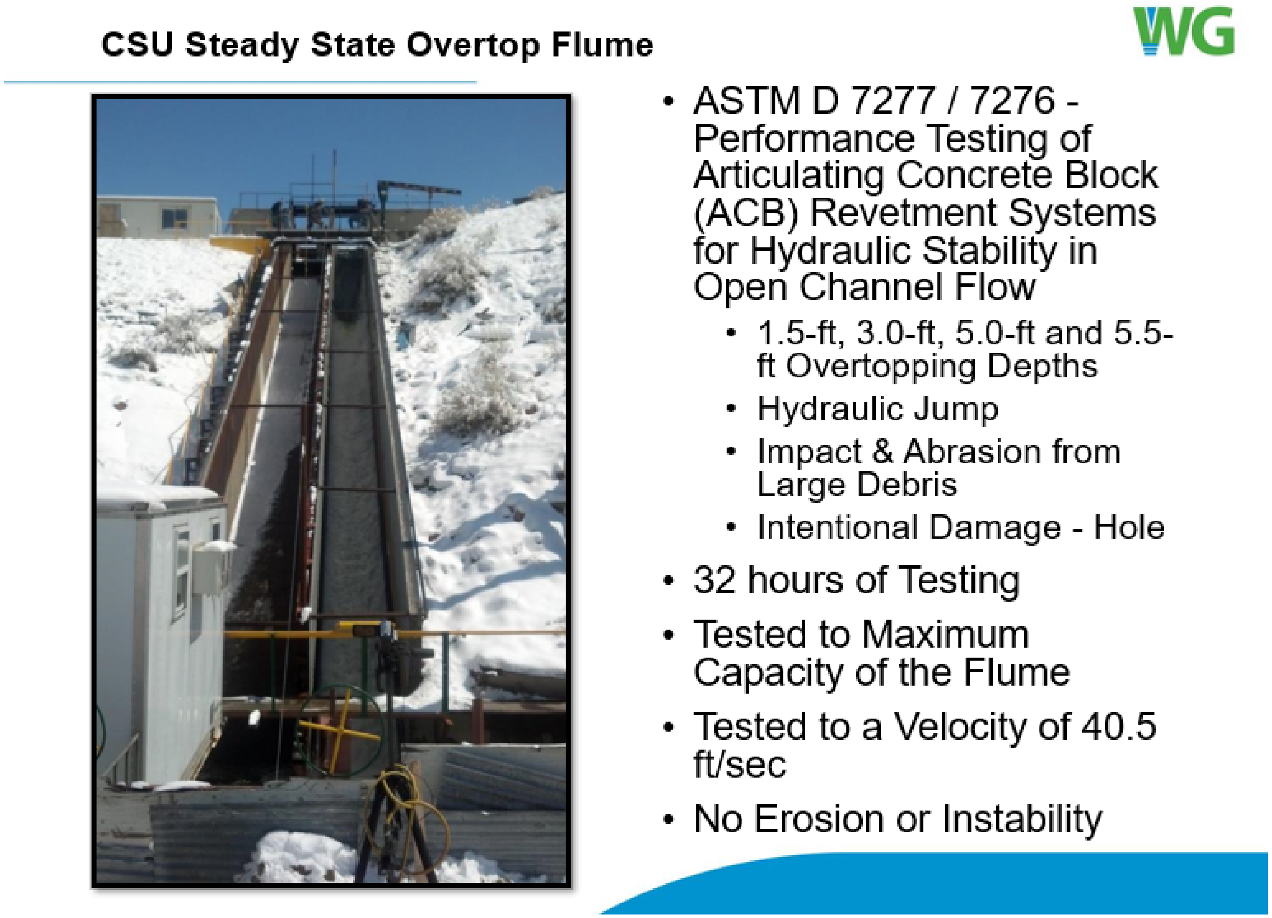
Synthetic Turf Revetments - Hydroturf



HydroTurf® CS Components

Source: Watershed Geo. Used with permission

Hydroturf Continued



CSU Steady State Overtop Flume

WG

- ASTM D 7277 / 7276 - Performance Testing of Articulating Concrete Block (ACB) Revetment Systems for Hydraulic Stability in Open Channel Flow
  - 1.5-ft, 3.0-ft, 5.0-ft and 5.5-ft Overtopping Depths
  - Hydraulic Jump
  - Impact & Abrasion from Large Debris
  - Intentional Damage - Hole
- 32 hours of Testing
- Tested to Maximum Capacity of the Flume
- Tested to a Velocity of 40.5 ft/sec
- No Erosion or Instability

Source: Watershed Geo. Used with permission

Takeaways

- Hire an experienced, qualified Engineer
- Pay attention to Manufacturers’ specifications and lab testing data
- Modification of a dam in Maryland requires a permit from Maryland Dam Safety
  - This includes drilling